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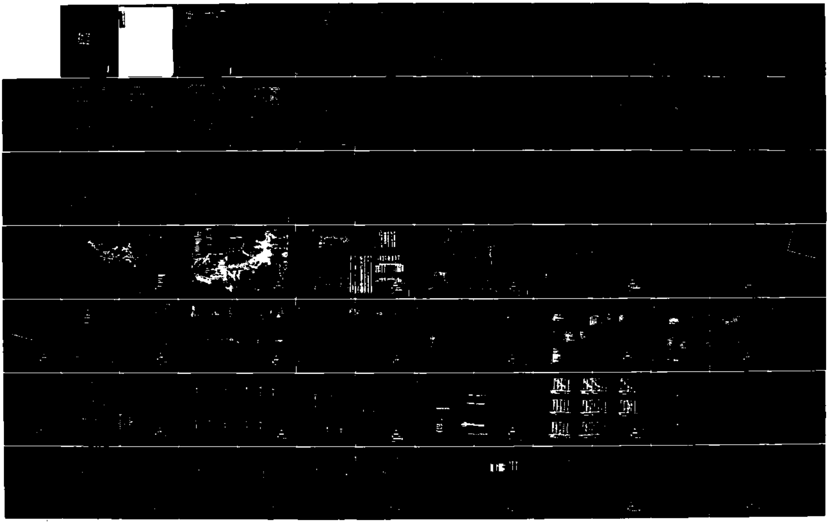

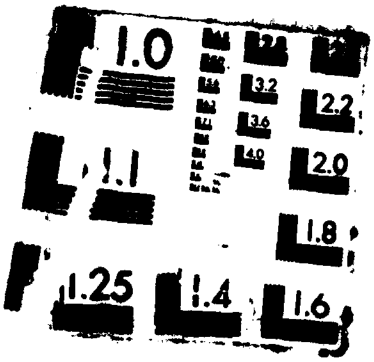
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The purpose of this Supplement No. 1 to the Smithville Lake, Embankment Criteria and performance report is to present the general plan used to improve the stability of the dam at high pool levels, which includes (1) investigations performed, (2) results tabulated, (3) remedial measures installed and (4) performance to date.

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OPERATION AND MAINTENANCE MANUAL

SMITHVILLE, LAKE
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APPENDIX V

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EMBANKMENT CRITERIA AND PERFORMANCE REPORT

SUPPLEMENT NO. 1

MARCH 1987

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DEPARTMENT OF THE ARMY
KANSAS CITY DISTRICT, CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

OPERATION AND MAINTENANCE MANUAL
SMITHVILLE LAKE
LITTLE PLATTE RIVER, MISSOURI

APPENDIX V

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

SUPPLEMENT NO. 1

LEFT ABUTMENT REMEDIAL MEASURES

Form 1473

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APPENDIX V

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

SUPPLEMENT NO. 1

LEFT ABUTMENT REMEDIAL MEASURES

INTRODUCTION

1. Purpose of Supplement. The purpose of this supplement is to present the general plan used to improve the stability of the dam at high pool levels. A brief description is included of the investigations performed, results, remedial measures installed, and performance to date. The results of the stability analyses conducted on the left abutment based on project performance since installation of the remedial measures is presented. Detailed descriptions of the field and laboratory investigations are found in the reports entitled 'Left Abutment Stability Report' and 'Left Abutment Seepage Report' contained in appendices A and B, respectively, of the supplement.

2. Impoundment history. Impoundment began in October 1979 but lake filling was delayed because of real estate acquisition problems. Multipurpose pool, at elevation 864.2, was first reached in June 1982. A pool at elevation 869.4 occurred in April 1983 and again in April 1984. The record high pool, at elevation 873.17, was reached on 16 and 17 October 1985. Flood control pool is at elevation 876.2, while spillway crest pool is at elevation 880.2.

3. Piezometric levels and seepage. During the fall of 1982, after the pool had attained multipurpose level, high piezometric response was noted in several of the piezometers downstream of the centerline. Comparisons between the observed and anticipated piezometric levels suggested the safety factor for the steady seepage case might be lower than the 1.6 shown in the embankment design memorandum. Close monitoring of the piezometric levels was recommended, particularly at station 110+00 where the piezometric level was about 3 feet above the ground surface. During April 1983 a seep area developed at the toe at station 110+00 with the pool at elevation 869.37, a record high pool at that time. Seepage was not enough to observe flowing quantities. In August 1983 an adjacent landowner, Roy Bowers, reported that a large wet area, located about 3,000 feet below the main dike, had developed on his property. Field reconnaissance revealed three general areas of seepage: (1) the downstream seep area at the toe of the left abutment; (2) the Bowers seep area; and (3) the dike seep area. These general areas are shown on plate 4. An investigation of seepage in the left abutment was then initiated. A brief overview of the investigation is presented below, a detailed description of these events are contained in appendices A and B.

4. Investigations and actions.

a. Initial. The initial investigation included the installation and monitoring of additional instrumentation devices and a review of the original design analysis. Fourteen piezometers and observation wells were installed in a line between the lake and the Bowers seep area and five piezometers were installed in the downstream seep area as shown on plates 5 and 7. Based upon

the observed data and a review of the original design stability analysis it was concluded that projected piezometric levels for the pool at spillway crest were higher, as much as 25 to 35 feet, than those assumed during design. Preliminary stability analysis indicated that the safety factor using the DM design strengths and the projected piezometric levels was well below the DM recommended 1.6.

b. Subsequent. Commencing in April 1984 a sequence of actions was begun to insure the safety of the structure while slope stability and seepage investigations were being conducted. A revised plan of lake releases was put into effect to reduce the probability of subjecting the dam to high pool levels and the frequency of monitoring the dam was increased for all pools above multipurpose. Because of immediate concerns regarding the stability of the dam at high pool levels an interim solution that would reduce piezometric levels by pumping from wells was implemented. After the wells were installed and operational the project was returned to a normal operating plan. Two inclinometers were installed in the most critical areas at stations 108+00 and 110+00 to monitor any movement. Ten relief drains were installed on the Bover's and adjacent Government properties to provide a measure of seepage control and to determine the effect on the piezometric surface. Four flowing test wells were subsequently installed at the toe of the embankment to evaluate whether pressure relief wells would provide a satisfactory permanent solution. During the installation of the wells and piezometers undisturbed sampling was being performed and a laboratory testing program being conducted. Of concern was the strength and continuity of a slickensided surface in a shale seam located near the top of bedrock. Appendices A and B contain additional detailed information regarding these investigations and actions.

c. Final. The installation of 13 pressure relief wells and a buried collector pipe in the left abutment was completed early in 1985. Plate 6 shows the location of these wells. With these wells controlling seepage and reducing uplift pressures the stability analysis had indicated that an adequate factor of safety would be obtained for the spillway crest steady seepage condition. A detailed description of the investigations regarding the stability analysis is contained in appendix A. Additional stability analysis performed after installation of the relief wells is presented in later paragraphs.

GEOLOGY

5. General. Smithville Lake is located near the southern limit of the Dissected Till Plains Section of the Central Lowlands Physiographic Province. Major topographic features are the maturely to submaturely developed valleys of the Little Platte River, Crows Creek, and Camp Branch. Drainage patterns typical of northern Missouri are developed on thick glacial deposits resulting in gently rolling topography. Bedrock exposures are not common but can occasionally be found along the bases of valley walls of major streams. Maximum relief in the area is about 160 feet.

6. Glacial history. Pleistocene glaciers extended into the northern region of Missouri approximately 750,000 years ago during the Kansan glacial episode and persisted for approximately 100,000 years. Glaciers may have also advanced into the area during the earlier Nebraskan episode. Both the Nebraskan and Kansan advances were from the north-northwest and are attributed

to the Iowa ice lobe from the Keewatin ice center in Canada. Since the same general regions were traversed during both episodes, the content of resultant drift materials is similar and difficult to distinguish. The southern limit of glaciation is generally recognized as being slightly south of, and approximately parallel to, the present course of the Missouri River.

Pleistocene ice sheets have been compared in size and extent to those of the Antarctic which have an average central thickness of about 6,500 feet. Estimated thicknesses of marginal masses are of the order of 1,600 feet. Glacial erosion was primarily by abrasion and quarrying whereby slabs of frozen ground were sheared from and dragged forward over nonfrozen ground. Magnitudes of erosion were dependent upon the thickness and velocity of the ice mass, the nature of materials incorporated into the basal ice, and the character of surfaces overridden. Glacial sediments include nonstratified till and, less frequently, fluvio-glacial deposits of stratified silts, sands, and gravels. Drift of variable thickness has been deposited upon essentially flat lying Pennsylvania bedrock and is the thickest in pre-Pleistocene topographic lows.

7. Overburden. Overburden in the vicinity of the dam is of three principal types; alluvium, glacial drift, and loess. Alluvium occupies the valleys of the Little Platte River and its tributaries and generally consists of lean and fat clays overlying clayey sands and sandy clays with minor amounts of basal gravel. Thicknesses range from 25 to 50 feet. Upland areas are deposits of glacial drift thinly mantled with loess. In the left abutment area, the drift ranges in thickness up to 85 feet and generally consists of 20 to 60 feet of till overlying 5 to 25 feet of coarser outwash sediments. Till, in general, is composed of unsorted, unconsolidated (geologically), nonstratified sediments deposited directly by and underneath glacial ice masses and consists of heterogeneous, random mixtures of clay, silt, sand, gravel, cobbles, and boulders. The overburden above approximately elevation 845 in the left abutment is predominantly lean clay glacial till with scattered gravel and cobbles and occasional isolated silty sand lenses. Below elevation 845, the material is much more heterogeneous with considerable lateral and vertical variation. Throughout most of the abutment area, the upper 11 to 20 feet of this lower unit is generally silt, however, silty clay or lean clay was encountered in some borings at this horizon. Below the silt zone, the material is coarser and consists of sand, gravel, and cobbles, generally with a significant amount of silt and clay. The coarser materials underlying the till are meltwater sediments deposited from advancing or retreating ice sheets. Loess overlying the till reaches a thickness up to 20 feet in the area. The maximum thicknesses occur on broad, gently sloping, interstream divides where erosion has been minimal.

8. Bedrock. Near surface bedrock strata are of the Pennsylvanian System, Lansing and Kansas City Groups and consist of alternating beds of shale and limestone. A geologic column for the left abutment is shown on plate 8. The essentially horizontal configuration of the left abutment bedrock surface is the result of a pre-Pleistocene stream channel trending generally east-west through the abutment. It is one of two major channels mapped in the reservoir area which are part of the ancestral Missouri River drainage system prior to the advance of Pleistocene glaciers. The other is located several miles upstream of the dam in the reservoir area. As ice masses traversed the area, existing sediments were scoured away and near-surface bedrock strata subjected to shear forces induced by ice thrusts.

9. Characterization of the left abutment foundation. The investigations more clearly defined a basal layer of glacially deposited materials that form a pervious zone beneath the entire left abutment. The basal layer, consisting of a heterogeneous mixture of silt and silty or clayey sands, varies in thickness from 40 feet beneath the upland portion of the abutment to less than 10 feet near the valleys. Plates 9 and 11 contain profiles and sections through the abutment and the embankment showing these trends. The thicker portions tend to be more gravelly and cobbly whereas sand becomes more predominate as the layer thins. Similar materials crop out in the bluffs along Crows Creek upstream of the dam. The natural pervious, in combination with the overlying lean clays and silts and underlying tight bedrock, forms a confined aquifer system that is recharged from the lake. The seep areas are characterized by a decreasing thickness of low permeability material with pockets or lenses of more pervious material extending to or near ground surface.

10. Seepage behavior. With impoundment the exposed basal material along Crows Creek was submerged, permitting saturation of the basal layer and subjecting the confined aquifer system to a hydrostatic pressure head corresponding to the pool level. The piezometric pressure gradient is relatively flat through the abutment, except for initial entrance pressure losses and near the seeps. As piezometric levels in the basal layer near the downstream base of the abutment increased to above ground level a sufficient vertical gradient was created to force seepage upwards through the thinning confining layer. Because a relatively impervious alluvium blocks this flow seepage quantities are low. Up to a 50 percent piezometric response to changes in the pool have been observed. A more detailed discussion of the seepage is contained in appendix B.

SEEPAGE CONTROL

11. General. The installation of the relief drains and relief wells was completed by early 1985. The investigation into the cause, extent, and chosen remedial solutions to the seepage and stability problems was briefly summarized in the previous paragraphs. The appendices contain reports that address these matters in greater detail. The following paragraphs discuss the performance of the remedial actions to date.

12. Bowers seep area. A total of 10 relief drains, 7 on the Bowers property and 3 on the adjacent Government land, were installed as shown on plate 7. Boring logs for the 10 drains are shown on plate 14. A temporary collector system consisting of an above ground plastic pipe currently connects relief drains 4, 5, 5A, 5B, 6, 7, and 8 while drains 1, 2, and 3 discharge onto the ground. The relief system on the Bowers property is estimated to be flowing at a rate of 30 gpm and has succeeded in significantly drying up the area. Plate 11 shows a profile through the Bowers seep area with the piezometric levels prior to and following drain installation. A buried collector system is to be installed after acquisition of the property and will result in a further lowering of the piezometric surface.

13. Downstream seep area. A total of 13 relief wells were installed in the left abutment as shown on plate 5. Boring logs and installation details are shown on plates 15, 16, and 17. Relief wells 3 through 11 discharge into a buried collector system that exits through a Parshall measuring flume into

the toe ditch. The collector system consists of a fabric lined trench with gravel surrounding a slotted, corrugated plastic pipe. Relief wells 1 and 2 discharge at the toe of the dam below the flume while wells 12 and 13, located on the downstream slope, discharge into the pervious drain. Figures 13 through 21 show the relief well flow versus pool for wells 3 through 11 and figure 22 shows the total combined flow of these wells. The collector system flows at a rate of 25 to 30 gpm which is generally about 5 gpm higher than that of the wells. This is attributed to any or all of the following: (1) infiltration of ground water into the collector system; (2) error in the calibration of the flume; or (3) well flow measurement errors. Flow from wells 12 and 13 are too small to measure but they have caused a reduction of the piezometric level in the immediate area. Figures 1 through 12 relate the pool elevation to the piezometric level in the left abutment. The plots show a fairly well defined relationship between the pool and the piezometric level for both prior to and following well installation. The result is that the wells have performed as anticipated and have effectively reduced the piezometric surface in the left abutment resulting in an improvement in the overall stability of the embankment. Periodic Inspection Report #5 contains detailed piezometric data obtained over the life of the project. Plate 26 shows the projected piezometric levels with and without the wells, at the record high pool, and projected levels at spillway crest.

14. Dike seep area. The projected piezometric levels at higher pools are above ground level, however, the thickness of the confining layer should prevent excessive seepage. No remedial measures were deemed necessary although some surface drainage may facilitate maintenance of the area for surveillance purposes.

STABILITY ANALYSIS

15. Design Memorandum No. 10 "Soil Data and Embankment Design" stability studies. Investigations for the DM included stability analysis for the left abutment steady seepage, rapid drawdown, and partial pool cases. Water pressures were computed to be hydrostatic below the saturation line. For the left abutment section the assumed saturation line under the downstream slope was in the foundation at elevation 825. High piezometric levels resulting from underseepage were not expected to be a problem "due to the thickness of the impervious foundation materials, the large amount of fines in the sands and gravels, and the scarcity of continuous pervious layers beneath the embankment." DM No. 10 plate A-37 summarizes design shear strengths and illustrates the critical shear surfaces.

16. Stability review of DM No. 10. An initial investigation of the left abutment stability was begun after seepage areas and high piezometric levels were observed. Additional piezometers were installed through the downstream slope and the original design stability analysis was reviewed. The investigation concluded that:

a. The piezometric level assumed for design in the left abutment was actually some 20 feet below the base of the horizontal pervious blanket in the area at station 110+00.

b. Projected piezometric levels for a spillway crest pool based on recorded data up to that time were about 25 to 35 feet above that assumed for design.

c. The higher than anticipated piezometric levels meant that available shear strength of the foundation shales become more critical.

d. The uppermost bedrock units in the left abutment have been exposed to erosional unloading and glacial loading. These factors, coupled with a nearly horizontal bedrock surface and essentially flat lying sedimentary rock units, suggested the possibility of the existence of a shear zone or zones near the bedrock surface.

e. Preliminary stability analysis conducted with the projected piezometric levels and Design Memorandum No. 10 design strengths showed the safety factor well below the desired 1.6 of the DM.

17. Left abutment stability investigation. Shale design strengths used in Design Memorandum No. 10 were obtained from samples of shale units of the right abutment, outlet works area, and valley. Additional sampling and testing for the left abutment was not done for the original design since the factor of safety for the left abutment appeared to be adequate. The shale shear strengths become more critical when higher than anticipated piezometric levels were recorded in the left abutment. Accordingly, a more comprehensive stability investigation was initiated. The investigation included additional sampling and testing of left abutment materials, particularly to determine if a weak zone or zones existed, a reanalysis of slope stability, and recommendations for remedial measures needed to insure the stability of the embankment. A detailed account of these results may be found in appendix A.

18. Shale seam in the Raytown limestone. Sampling efforts were initially directed towards obtaining samples for testing the Raytown limestone-Muncie Creek shale contact. However, core samples in the Raytown limestone revealed slickensided planes in a soft shale seam located in the lower part of the bed. The seam is about 0.4 to 0.5 feet thick, approximately 1.5 feet above the Muncie Creek contact, and is persistent throughout the abutment. Apparent shear planes were observed in every sample recovered from the shale seam. Several thin shale partings are present above the seam but not continuous. The shale seam rather than the Raytown-Muncie Creek contact was determined to be the material with the least resistance to shearing forces and is the critical material for stability considerations. Testing of the seam yielded a residual strength with $\tan \phi = 0.13$ ($\phi = 7.4^\circ$) and a peak strength with c=250 psf, $\tan \phi = 0.268$ ($\phi = 15^\circ$).

19. Raytown limestone. The Raytown limestone was assumed to have a vertical joint and was assigned a resisting shear strength of zero. This assumption is consistent with Design Memorandum No. 10. Higher strengths for the limestone were available but were not used in the analysis since it would require thin, partially weathered, jointed limestone to carry very large forces.

20. Lower foundation overburden material. Design strengths used in Design Memorandum No. 10 for left abutment overburden material were obtained from tests run on lean and fat clays. As noted previously additional exploratory borings through the left abutment foundation overburden consistently revealed a coarse-grained pervious layer present above bedrock. As a result, it was believed reasonable to increase the S strength of the pervious layer to c=0.0, $\tan \phi = 0.577$ ($\phi = 30^\circ$).

21. Material strength approaches. Stability analyses for Design Memorandum No. 10 and preliminary stability analyses for the investigation were conducted using two different strength approaches: (1) peak strengths were used along the entire critical shear surface, and (2) peak design strengths were used along the failure surface in the active and passive wedges, and the residual shear strength of the shale was used in the central block portion of the sliding surface. The use of the residual strength was considered to be overly conservative since it assumed the lowest possible strength. The use of peak strengths for the stability analysis was believed to be unconservative due to strain incompatibilities between a shear zone in the shale, the foundation overburden, and the embankment. With small strains the peak strength could be developed in the shale before the peak strengths could be developed in the embankment and overburden materials.

22. Final material strength approach. For the interim and final analyses of the investigation a third approach was considered which accounted for strain incompatibilities. Peak strengths were used for material in the active wedge portion of the critical shear surface and in the shale seam, but not in the passive wedge since relative large displacements would be required to develop full passive resistance. It was considered reasonable to use strengths in the passive wedge (foundation overburden) which correspond to 0.5 percent strain development. Strains somewhat larger than this were required to develop the peak strength in tests on the slickensided shale surface.

23. Estimation of piezometric surface. The final stability analysis of the investigation consisted of locating the most critical shear surface of the left abutment for the steady seepage case at spillway crest (elevation 880.2). Stability analyses were considered with and without pressure relief wells at the toe of the embankment. Piezometric surfaces were determined by projecting piezometer levels for a pool at spillway crest and subtracting drawdown levels observed after installation of the four test relief wells at the toe of the embankment near station 110+00.

24. Method of analyzing stability. The stability analysis was conducted with the hand wedge method in accordance with EM 1110-2-1902 (April 1970) and with a computer program, SLOPE8R, developed at the University of California Berkeley. SLOPE8R uses Spencer's procedure to calculate the factor of safety for specified noncircular shear surfaces. In all cases, the factor of safety by the hand wedge method was lower than those obtained by SLOPE8R because the side force inclinations computed by SLOPE8R were greater than that assumed in the hand wedge analysis. When the same side force inclination was used in both the hand wedge analysis and SLOPE8R the computed factors of safety were similar. The results of the stability analyses performed at station 110+00 are as follows:

	Safety Factor	
	Hand Wedge	SLOPE8R
Spillway Crest Pool (without relief wells)	1.25	1.47
Spillway Crest Pool (with relief wells)	1.30	1.53

25. Recommended remedial measures. As a result of the stability analysis, it was concluded that the installation of pressure relief wells at the toe of embankment would provide an adequate factor of safety for steady seepage conditions with the pool at spillway crest. Recommendations included the installation of additional pressure relief wells and a buried collector system.

26. Current stability investigation. The stability of the left abutment following installation of remedial measures, consisting of 13 relief wells and a collector system, has been reviewed. Uplift pressures in the lower foundation materials are projections based on piezometric levels observed since installation of remedial measures.

27. Method of analyzing stability. A computer program entitled UTEXAS2 (September 1985) was used to analyze the slopes for the stability investigation. The program allows the user to select either a circular or noncircular shear surface with or without a search option to locate the shear surface with the minimum factor of safety. Reservoir pools are represented as an external surface load. The program, though relatively new, has been manually checked by the hand wedge method several times to assure its accuracy. It was also checked against previous SLOPE8R results. For this stability investigation, a noncircular shear surface with the search option was chosen. For each analysis the factor of safety against sliding was determined using Spencer's procedure.

28. Determination of the critical section. The first step of the stability investigation was to determine which cross-section of the left abutment would be the most critical. Cross-sections of stations 110+00, 111+00, 112+00, 113+00, and 114+00 were drawn from construction surveys, drill logs of exploratory borings, and piezometer and relief well installations. The slope height above natural ground decreased and the thickness of the passive wedge increased as the stations increased. The piezometric level does not increase significantly upstation. Therefore, based upon visual inspection, the cross-section at station 110+00 was determined to be the most critical.

29. Profiling the critical section. The embankment at station 110+00 consists of berm fill and compacted fill materials. Seepage through the embankment is intercepted by an inclined pervious wick which conducts it to the downstream toe of the embankment. Foundation overburden generally consists of two materials, finer grained upper overburden material and a lower coarser grained material. Bedrock under most of the embankment at station 110+00 is the Raytown limestone member which contains the soft shale seam. Raytown limestone is shown to extend 267 feet upstream from the centerline of the embankment although the scarcity of borings on the upstream side of the left abutment makes it difficult to determine the exact extent of the Raytown limestone. However, a drill log detailing the installation of piezometer P-110-2 (station 110+00, range 267 US) lists Muncie Creek shale as bedrock material; the Raytown limestone was not encountered. For the stability investigation, the Raytown limestone member containing the soft shale seam was conservatively assumed to terminate 267 feet upstream from the centerline of the embankment.

30. Shear strengths of materials. Shear strength parameters for the berm fill, compacted fill, and upper foundation overburden material were taken from Design Memorandum No. 10, "Soil Data and Embankment Design." Shear strength parameters for the lower foundation overburden (coarser material), the Raytown limestone, and the shale seam were obtained from the Left Abutment Stability Report (July 1984) and are referenced in appendix A.

31. Shear strength envelopes. For steady seepage and partial pool cases the shear strength parameters for the berm fill, compacted fill, and upper foundation overburden materials were defined by a nonlinear (S,(R+S)/2) envelope. The minimum nonlinear (S,R) envelope was used to define the shear strengths of these materials for the rapid drawdown case. The shear strength parameters for the lower (coarse) foundation overburden material, the Raytown limestone, and the shale seam remained the same for each analyses. The peak "S" (drained) strength was used for all cases for the coarse foundation overburden material, the residual "S" strength for the shale seam.

32. Determination of water pressures. For steady seepage conditions, the pore pressures acting on materials comprising the left abutment were determined from two piezometric surfaces. The pore pressures acting on the berm fill and compacted fill materials were defined by a line of seepage through the embankment. The line is assumed to extend horizontally at the pool elevation through the embankment to the inclined pervious wick. The wick and the horizontal pervious blanket conduct the seepage to the downstream toe of the embankment. Pore pressures acting on the lower (coarser) foundation overburden material, the Raytown limestone, and the shale seam were determined from water levels measured in a line of piezometers located at station 110+00. The corresponding piezometric surface for each pool elevation was projected from recorded levels read in each piezometer after installation of remedial measures. Pore pressures acting on the upper foundation overburden material were calculated by interpolating and averaging pore pressure values from the line of seepage and from pressures in the lower foundation overburden material.

33. Unit weights of materials. The unit weights used for the embankment and foundation materials are shown on plate No. 28. Unit weights for the berm fill, compacted fill, and upper foundation overburden materials were considered to be moist above the assumed line of seepage and saturated below. Saturated unit weights were used for the lower coarser foundation overburden material. The Raytown limestone and the shale seam were given unit weights of 140 pcf.

34. Cases considered for the stability analyses. This stability investigation considered both the upstream and downstream slopes of the embankment at station 110+00. The downstream slope was analyzed for a steady seepage condition with the pool at spillway crest (El. 880.2). The upstream slope was analyzed for steady seepage partial pool conditions and for a rapid drawdown from spillway crest pool to multipurpose pool (El. 864.2).

35. Partial pool case. A wide range of pool elevations was considered for the partial pool case. For a "no" pool condition the saturation line was assumed to be horizontal at an elevation of 820 feet. This places the piezometric level at the top of the lower (coarser) foundation overburden material for most of the section at station 110+00. This elevation for the

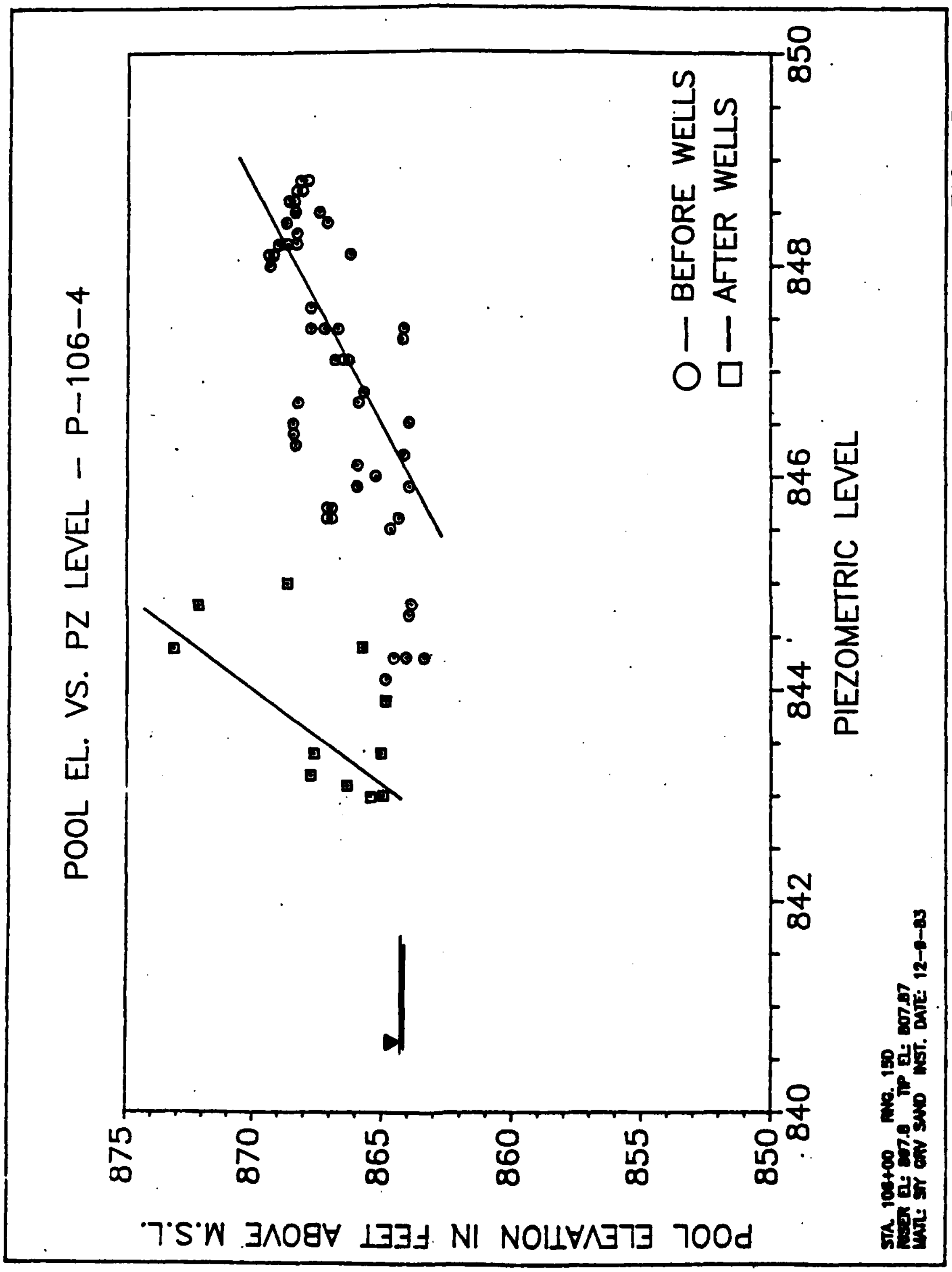
saturation line was determined from recorded piezometer levels after the embankment was constructed to its final height but before a measured pool elevation was recorded. The highest pool elevation considered was spillway crest at an elevation of 880.2 feet. The most critical failure surface for the partial pool analysis occurred at a pool elevation of 845 feet with a corresponding factor of safety of 1.37. The critical shear surface and the computed factor of safety for each considered pool elevation is shown on plate No. 29.

36. Rapid drawdown case. For the rapid drawdown analysis the pool was assumed to drop from spillway crest (El. 880.2) to multipurpose pool (El. 864.2). During rapid drawdown it was assumed that the drawdown occurred instantaneously and that no pore pressure dissipation occurred during drawdown. Therefore, the line of seepage through the embankment intercepted the upstream slope of the embankment at multipurpose pool and followed the upstream slope of the embankment to spillway crest pool. At this point the line extended horizontally to the inclined pervious wick. Water pressures acting on the lower foundation overburden and bedrock materials corresponded to the projected piezometric surface for spillway crest. Water pressures acting on upper foundation overburden material were calculated by interpolating pressures from the assumed line of seepage and from the piezometric surface. As previously mentioned, the shear strengths for the berm fill, compacted fill, and upper overburden materials were determined by the composite (S,R) envelope. Shear strengths for the remainder of materials remained the same as for the other stability analyses. The minimum factor of safety for the rapid drawdown case was calculated to be 1.30. The critical shear surface is illustrated on plate No. 30.

37. Steady seepage case. The downstream slope stability was analysed for a steady seepage condition with the pool at spillway crest (El. 880.2). Assigning the residual strength to the shale seam was considered for the previous stability analysis but not included because it was believed to be an overly conservative approach. In this investigation the residual strength was assigned to the shale seam but the required factor of safety lowered. A minimum factor of safety equal to 1.0 was sought. For the steady seepage case the calculated factor of safety was 1.00. Previously using peak strengths for the shale, reduced strengths in the other zones for strain compatibility, an estimated piezometric reduction from the relief wells, a factor of safety of 1.30 was obtained. The critical shear surface is illustrated on plate No. 30.

FIGURES

[illegible]



POOL EL. VS. PZ LEVEL – P–106–4
POOL ELEVATION IN FEET ABOVE M.S.L.
PIEZOMETRIC LEVEL
○ — BEFORE WELLS
□ — AFTER WELLS
875
870
865
860
855
850
840
842
844
846
848
850
FIGURE NO. 1

Figure No. 1

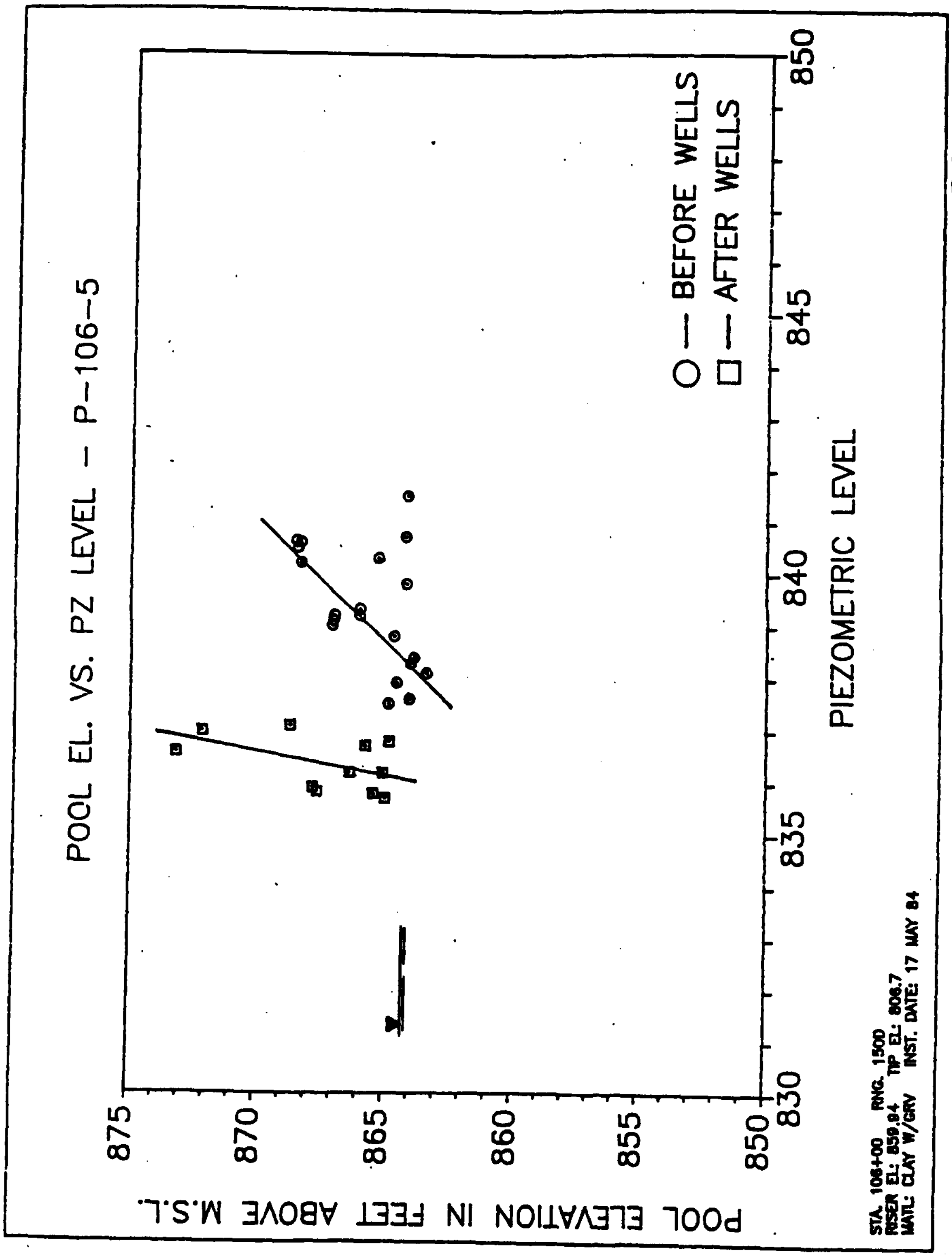

POOL EL. VS. PZ LEVEL — P-106-5
POOL ELEVATION IN FEET ABOVE M.S.L.
PIEZOMETRIC LEVEL
○ — BEFORE WELLS
□ — AFTER WELLS
875
870
865
860
855
850
830
835
840
845
850
STA. 106+00 RNG. 1500
RISER EL: 859.94 TIP EL: 806.7
MAT'L: CLAY W/GRV INST. DATE: 17 MAY 84
FIGURE NO. 2

Figure No. 2

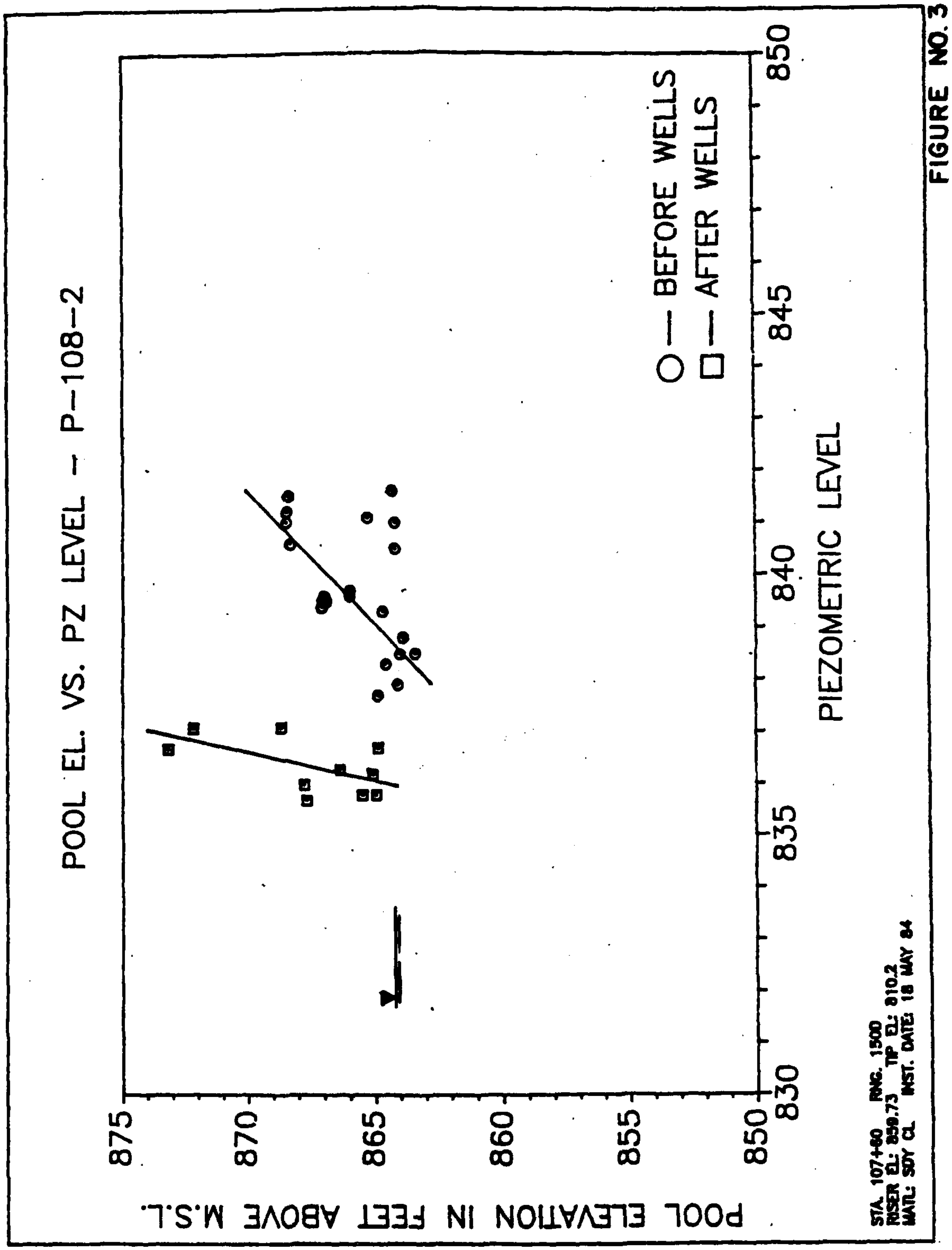

POOL EL. VS. PZ LEVEL — P—108—2
O— BEFORE WELLS
□— AFTER WELLS
PIEZOMETRIC LEVEL
POOL ELEVATION IN FEET ABOVE M.S.L.
830
835
840
845
850
850
855
860
865
870
875
FIGURE NO. 3
STA. 107+60 RNG. 1500
RISER EL: 859.73 TIP EL: 810.2
MAT'L: SDY CL INST. DATE: 18 MAY 84

Figure No. 3

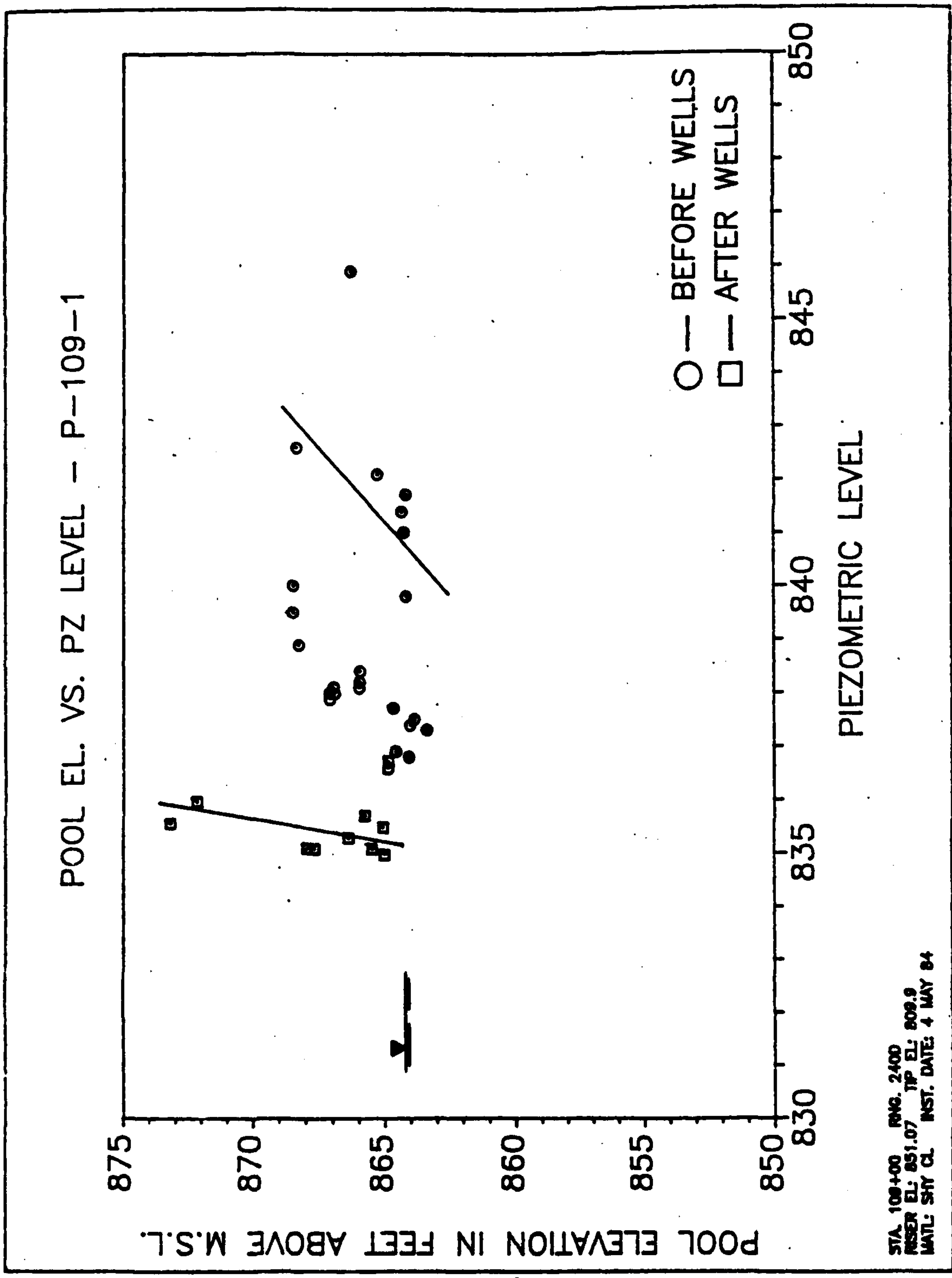
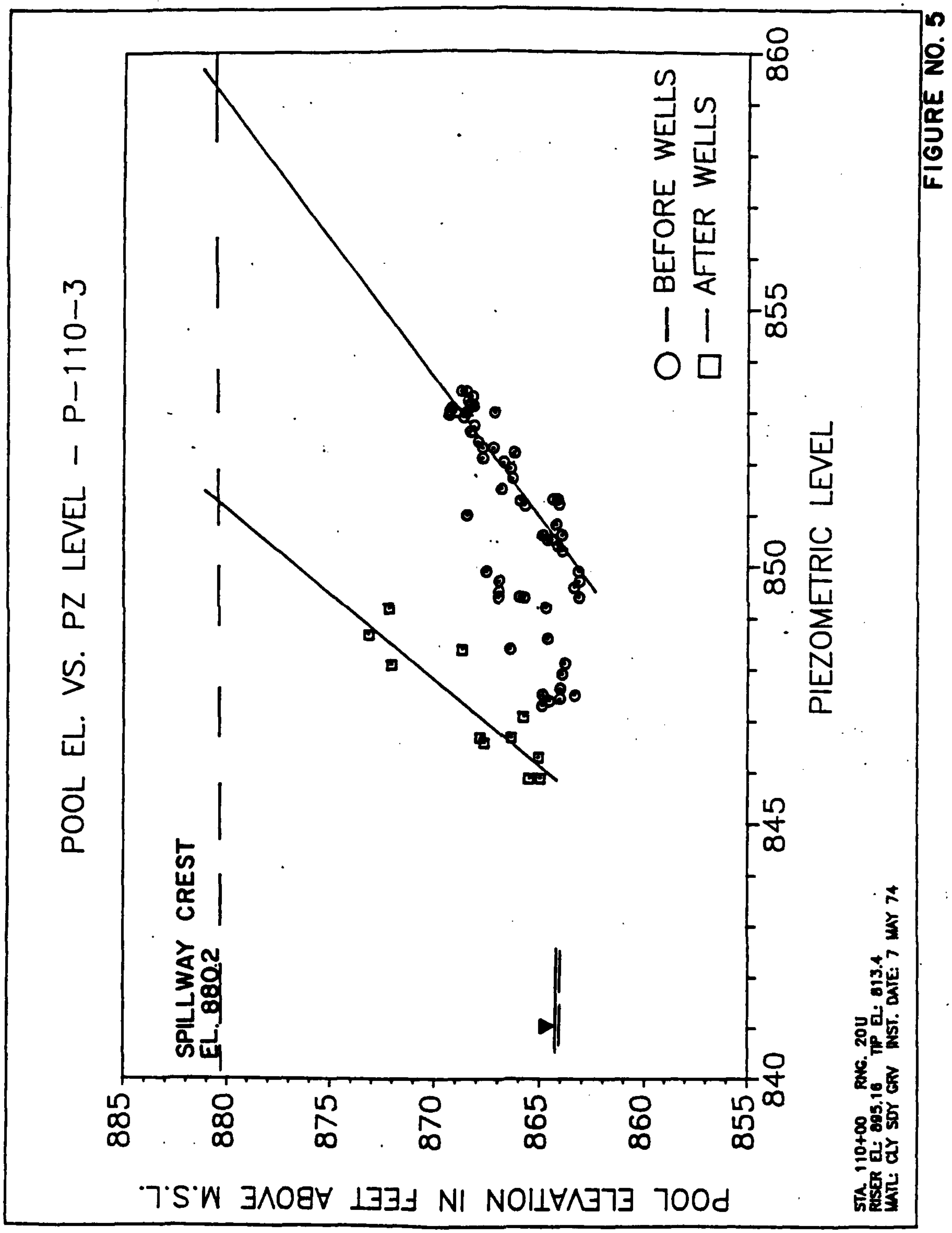


FIGURE NO. 4

Figure No. 4



POOL EL. VS. PZ LEVEL - P-110-3
POOL ELEVATION IN FEET ABOVE M.S.L.
PIEZOMETRIC LEVEL
885
880
875
870
865
860
855
840
845
850
855
860
SPILLWAY CREST
EL. 880.2
○ — BEFORE WELLS
□ — AFTER WELLS
STA. 110+00 RNG. 20U
RISER EL: 865.16 TIP EL: 813.4
MAT'L: CLY SDY GRV INST. DATE: 7 MAY 74
FIGURE NO. 5

Figure No. 5

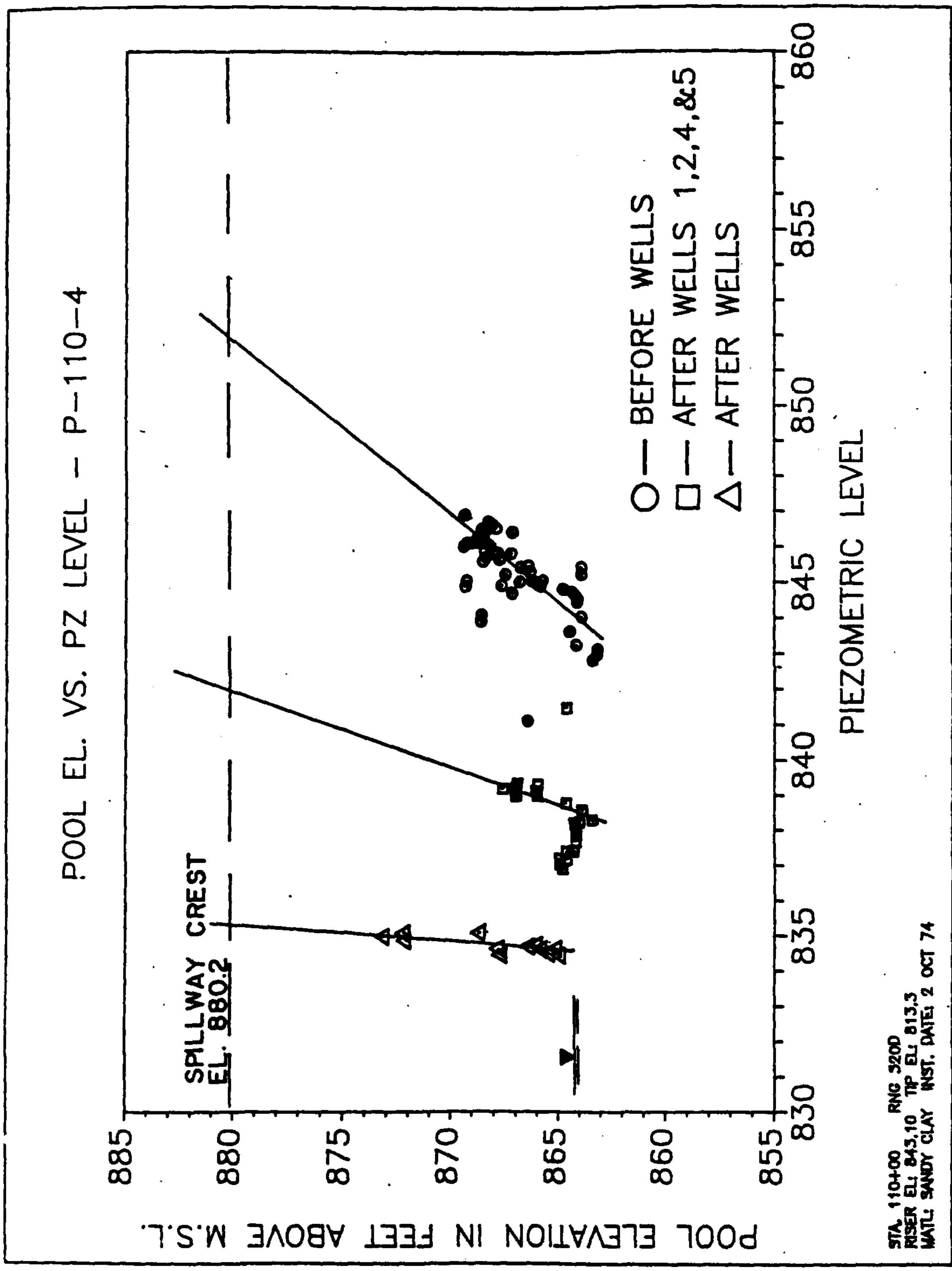


FIGURE NO. 6

Figure No. 6

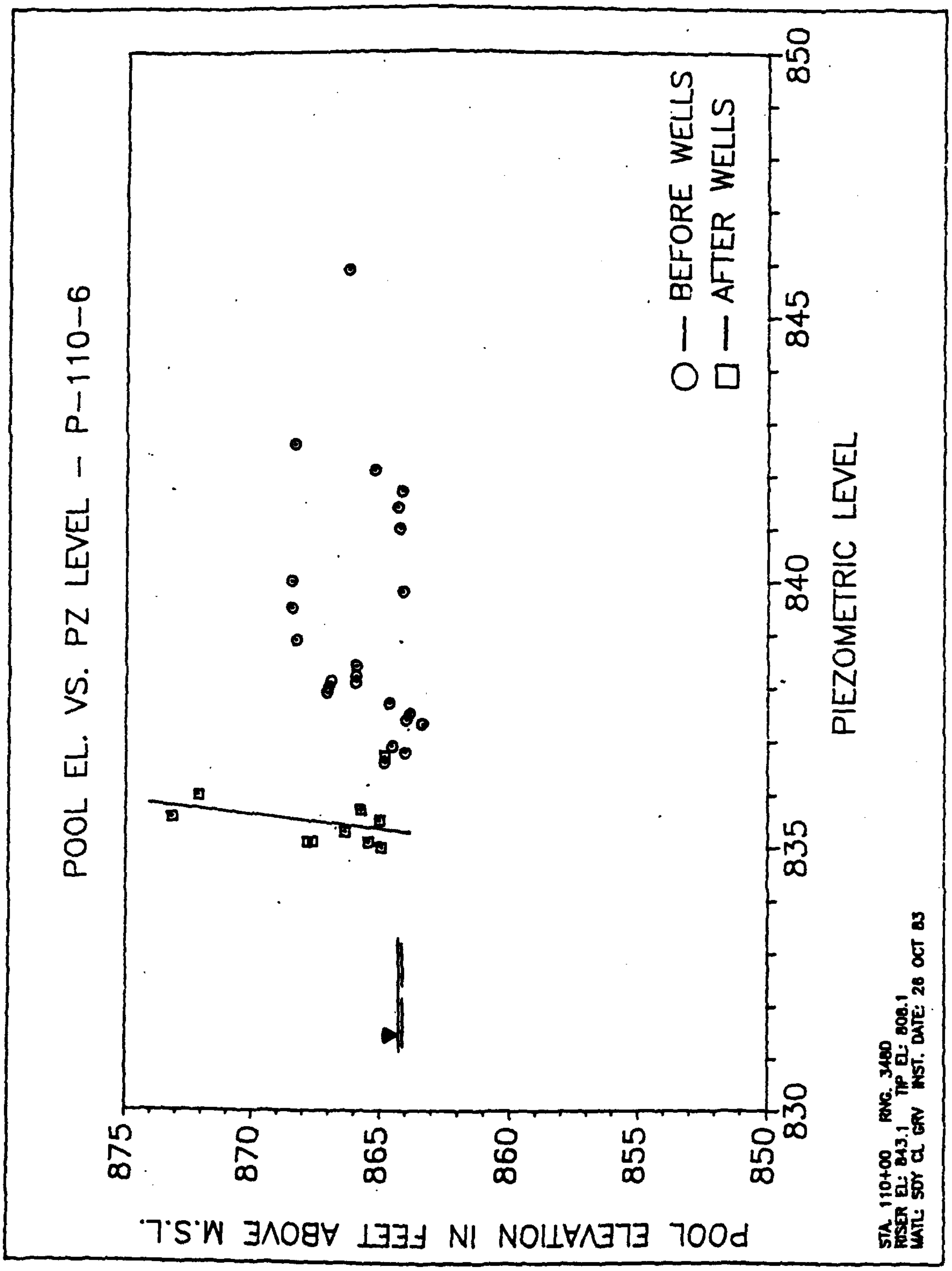


FIGURE NO. 7

Figure No. 7

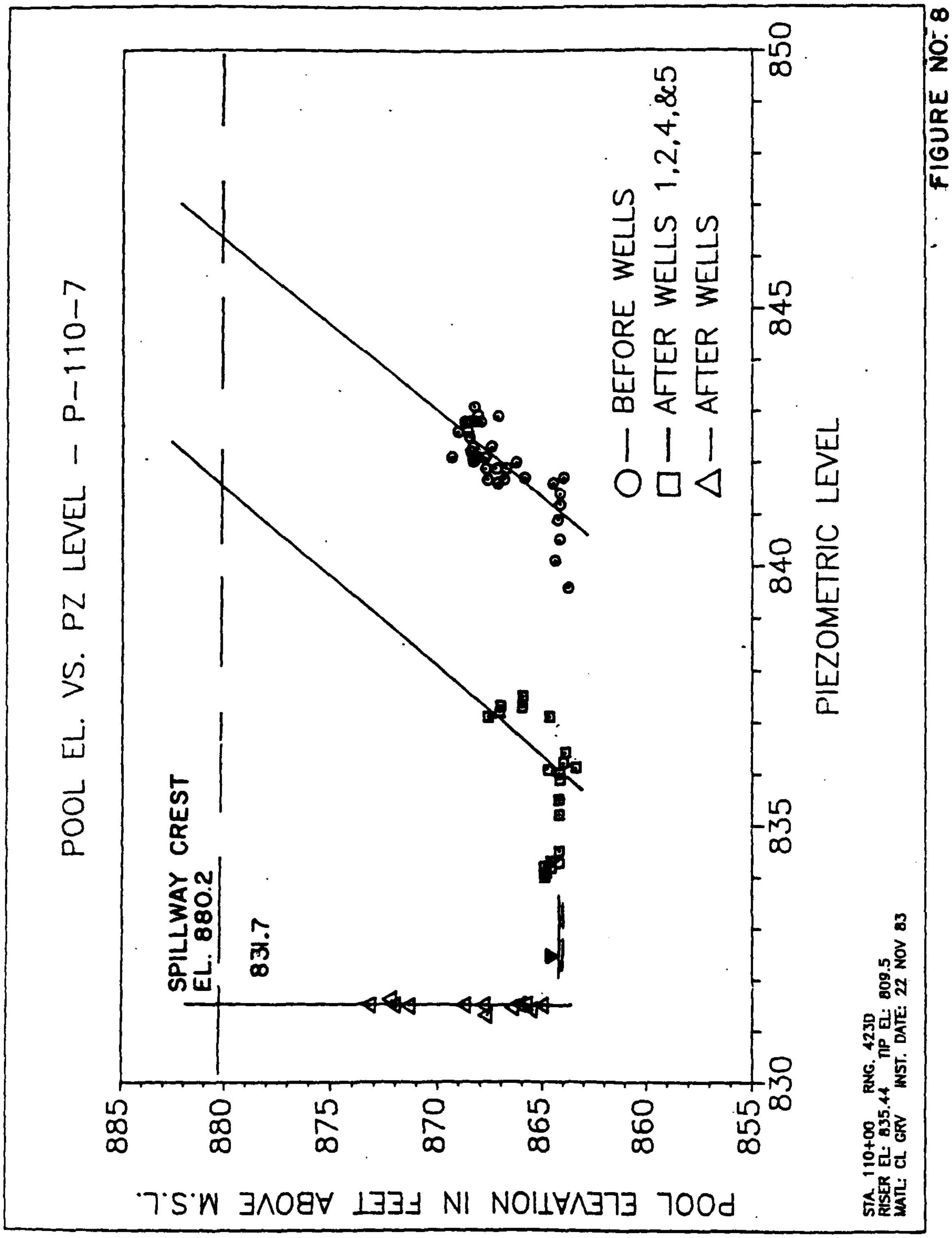
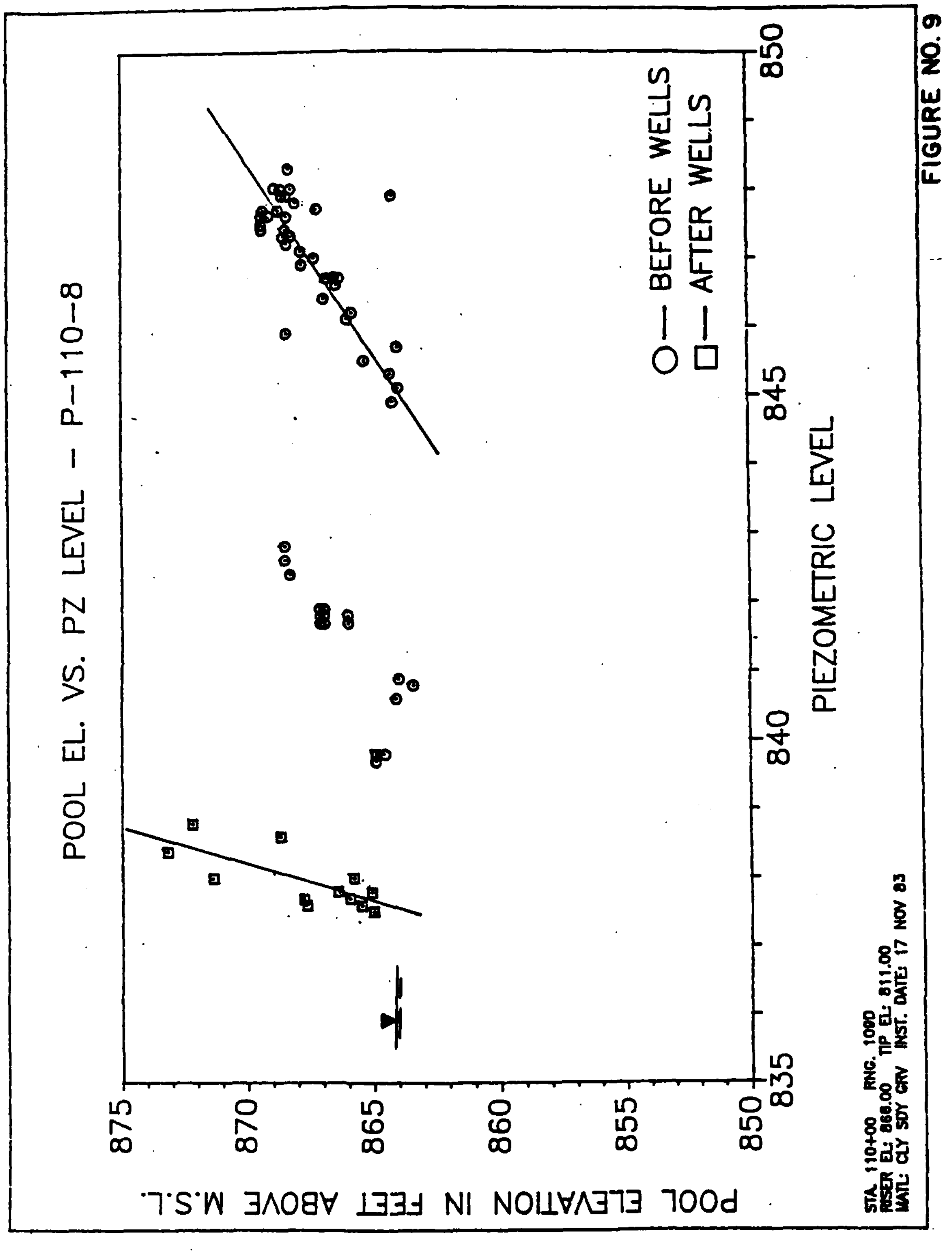

POOL EL. VS. PZ LEVEL – P-110-7
SPILLWAY CREST
EL. 880.2
831.7
POOL ELEVATION IN FEET ABOVE M.S.L.
885
880
875
870
865
860
855
830
835
840
845
850
PIEZOMETRIC LEVEL
○— BEFORE WELLS
□— AFTER WELLS 1,2,4,&5
△— AFTER WELLS
STA. 110+00 RNG. 423D
RISER EL: 835.44 TIP EL: 809.5
MATL: CL GRV INST. DATE: 22 NOV 83
FIGURE NO. 8

Figure No. 8



POOL EL. VS. PZ LEVEL - P-110-8
POOL ELEVATION IN FEET ABOVE M.S.L.
PIEZOMETRIC LEVEL
875
870
865
860
855
850
835
840
845
850
○— BEFORE WELLS
□— AFTER WELLS
FIGURE NO. 9
STA. 110+00 RNG. 109D
RISER EL: 866.00 TIP EL: 811.00
MAT'L: CLY SDY GRV INST. DATE: 17 NOV 83

Figure No. 9

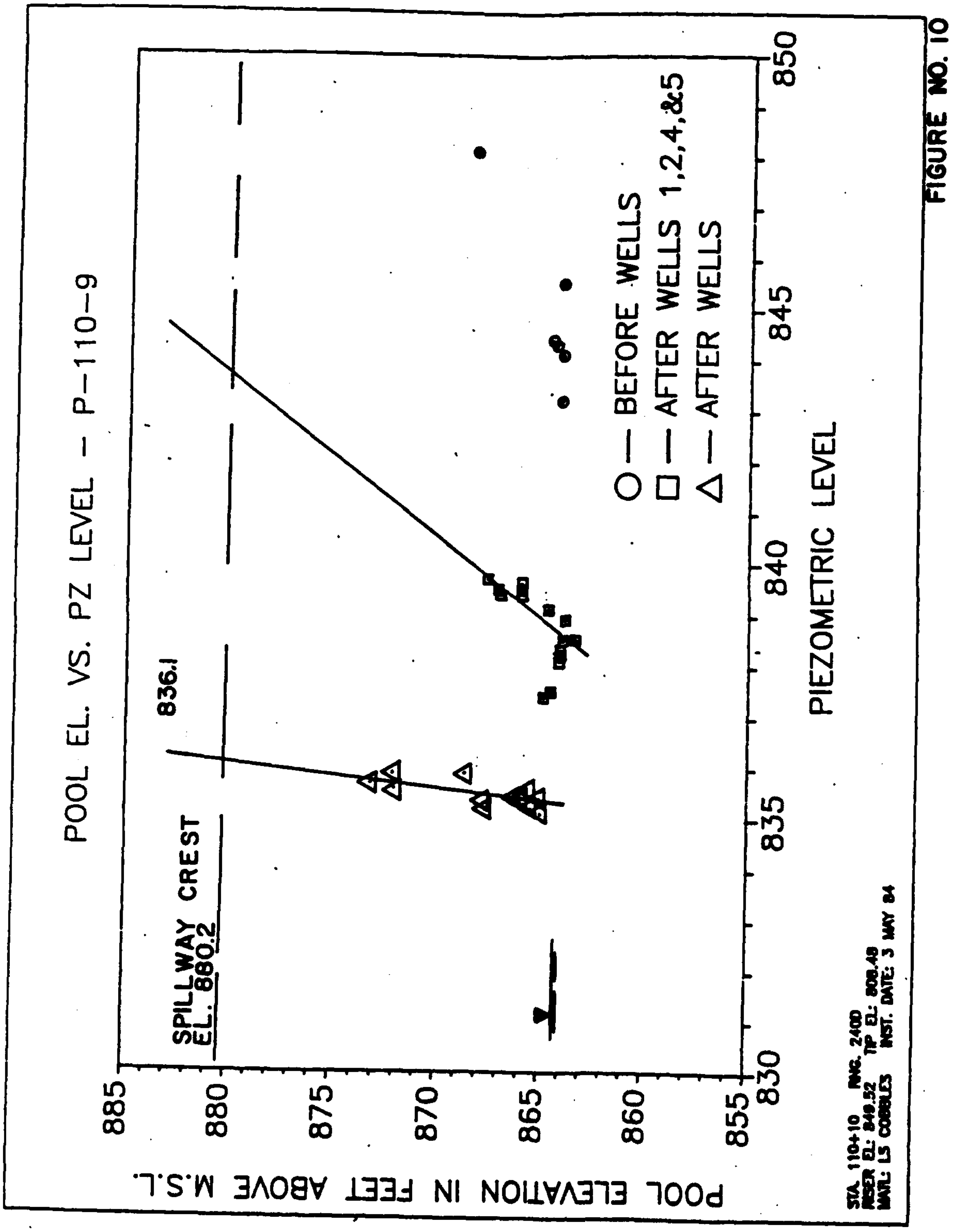

POOL EL. VS. PZ LEVEL – P–110–9
POOL ELEVATION IN FEET ABOVE M.S.L.
PIEZOMETRIC LEVEL
885
880
875
870
865
860
855
830
835
840
845
850
SPILLWAY CREST
EL. 880.2
836.1
○ — BEFORE WELLS
□ — AFTER WELLS 1,2,4,&5
△ — AFTER WELLS
FIGURE NO. 10

Figure No. 10

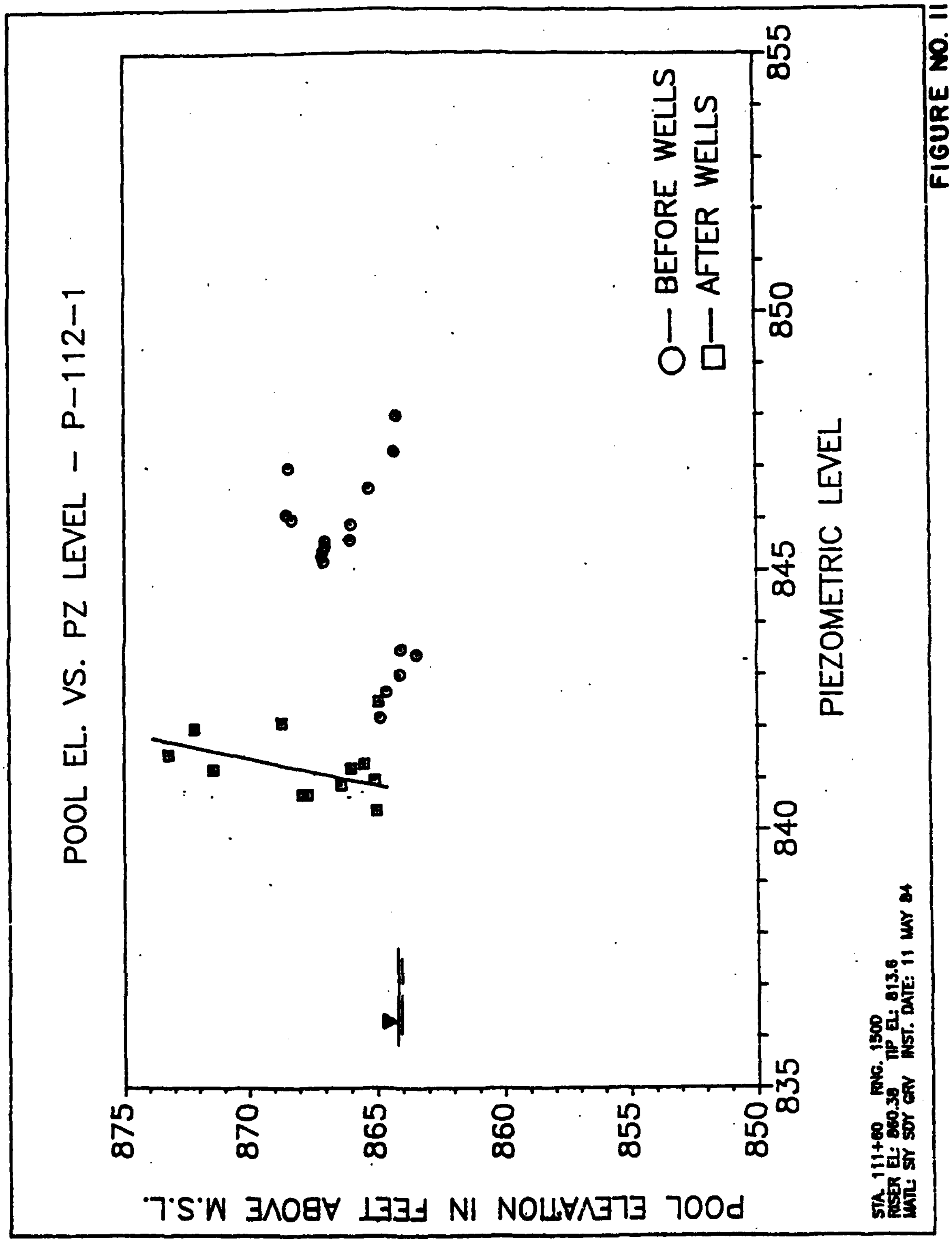

POOL EL. VS. PZ LEVEL – P–112–1
BEFORE WELLS
AFTER WELLS
PIEZOMETRIC LEVEL
POOL ELEVATION IN FEET ABOVE M.S.L.
835
840
845
850
855
850
855
860
865
870
875
FIGURE NO. 11
STA. 111+60 RNG. 1500
RISER EL: 860.38 TIP EL: 813.6
MATL: SIY SDY GRV INST. DATE: 11 MAY 84

Figure No. 11

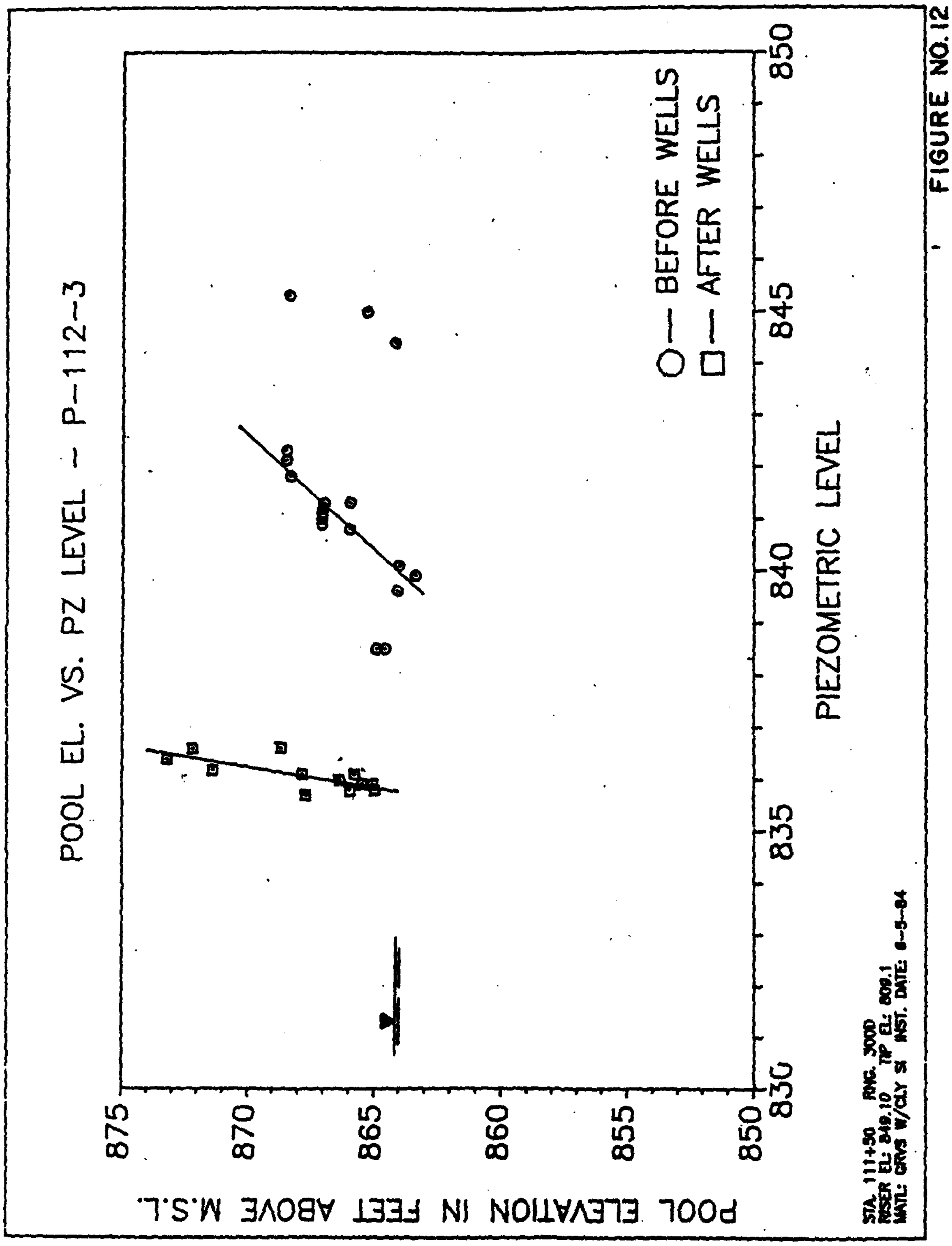

POOL EL. VS. PZ LEVEL - P-112-3
○— BEFORE WELLS
□— AFTER WELLS
PIEZOMETRIC LEVEL
830
835
840
845
850
POOL ELEVATION IN FEET ABOVE M.S.L.
850
855
860
865
870
875
FIGURE NO.12

Figure No. 12

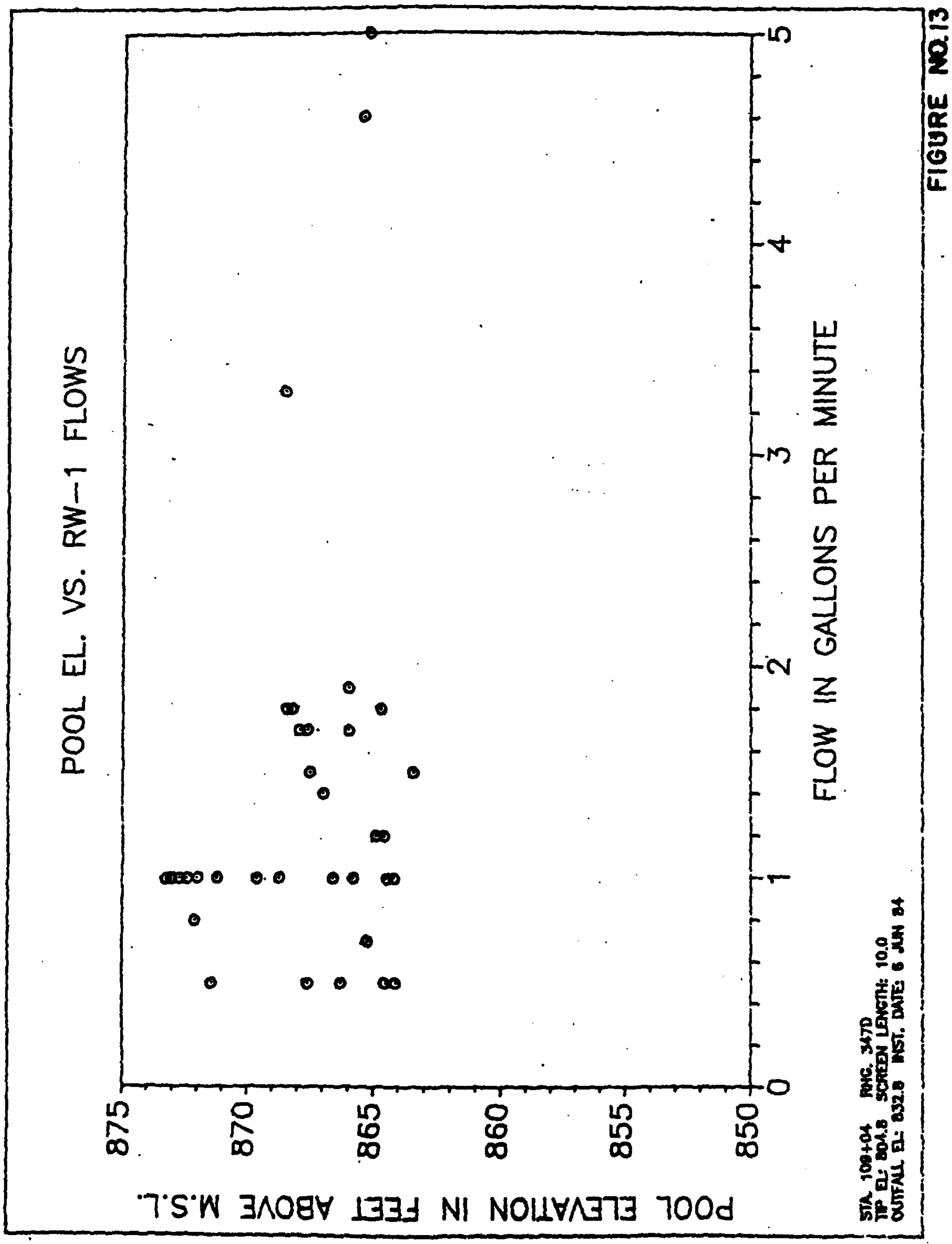

POOL EL. VS. RW-1 FLOWS
POOL ELEVATION IN FEET ABOVE M.S.L.
875
870
865
860
855
850
FLOW IN GALLONS PER MINUTE
0
1
2
3
4
5
STA. 109+04 RNG. 347D
TIP EL: 804.8 SCREEN LENGTH: 10.0
OUTFALL EL: 832.8 INST. DATE: 6 JUN 84
FIGURE NO.13

Figure No. 13

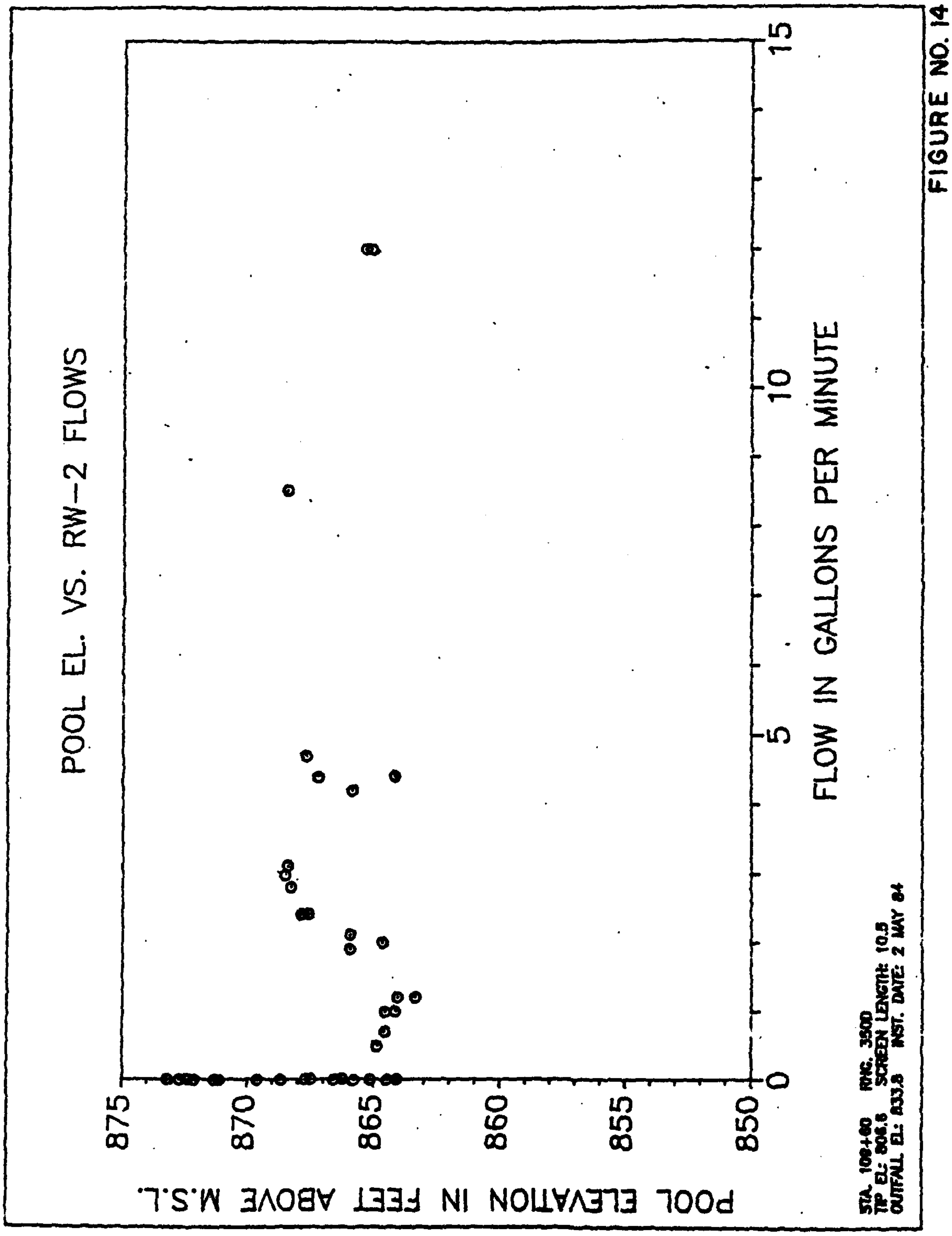

POOL EL. VS. RW-2 FLOWS
POOL ELEVATION IN FEET ABOVE M.S.L.
875
870
865
860
855
850
0
5
10
15
FLOW IN GALLONS PER MINUTE
STA. 108+60 RWC. 3500
TIP EL: 806.5 SCREEN LENGTH: 10.5
OUTFALL EL: 833.8 INST. DATE: 2 MAY 84
FIGURE NO. 14

Figure No. 14

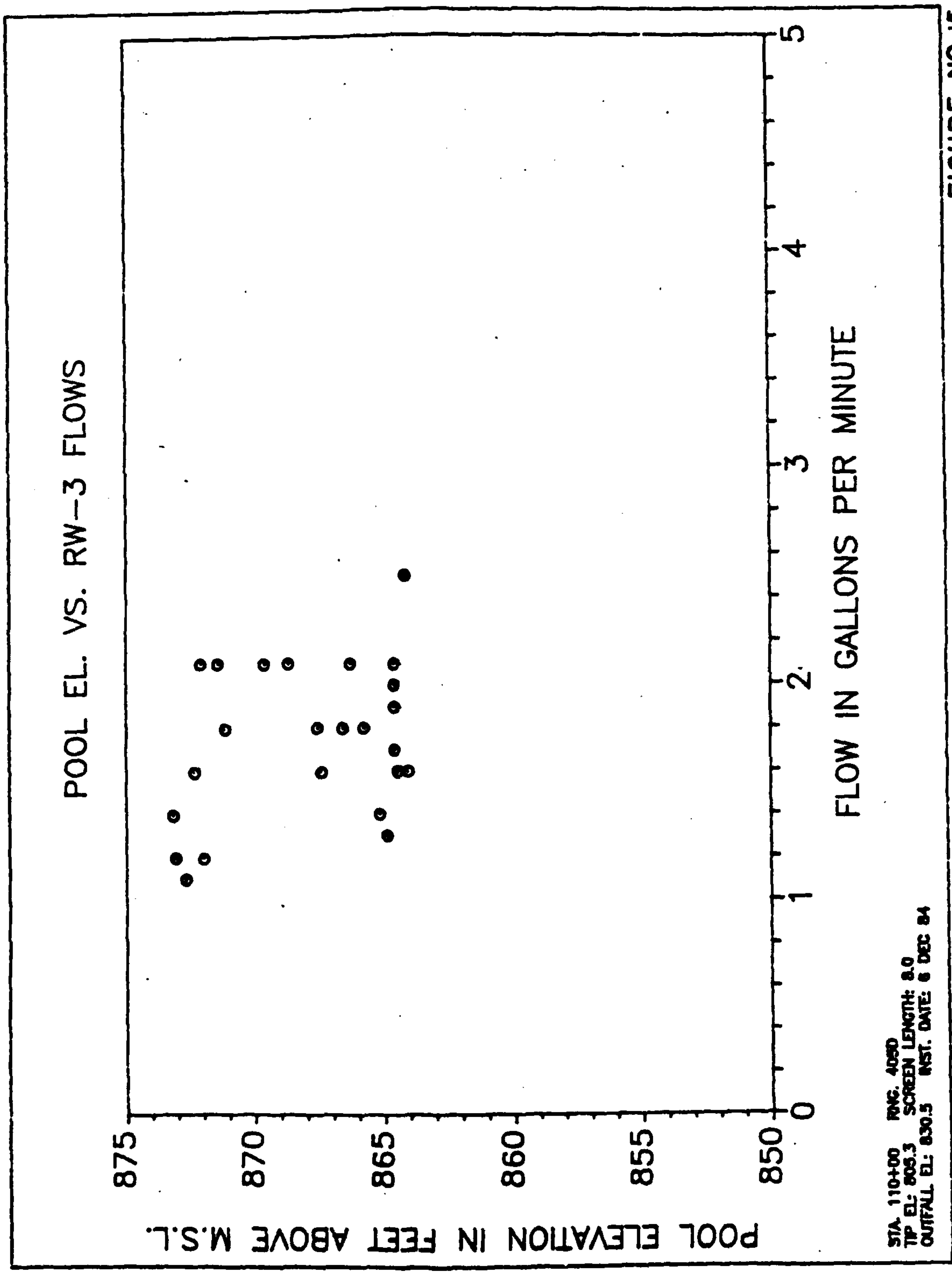

POOL EL. VS. RW-3 FLOWS
POOL ELEVATION IN FEET ABOVE M.S.L.
FLOW IN GALLONS PER MINUTE
875
870
865
860
855
850
0
1
2
3
4
5
STA. 110+00 RNG. 405D
TIP EL: 805.3 SCREEN LENGTH: 8.0
OUTFALL EL: 830.5 INST. DATE: 6 DEC 84
FIGURE NO.15

Figure No. 15

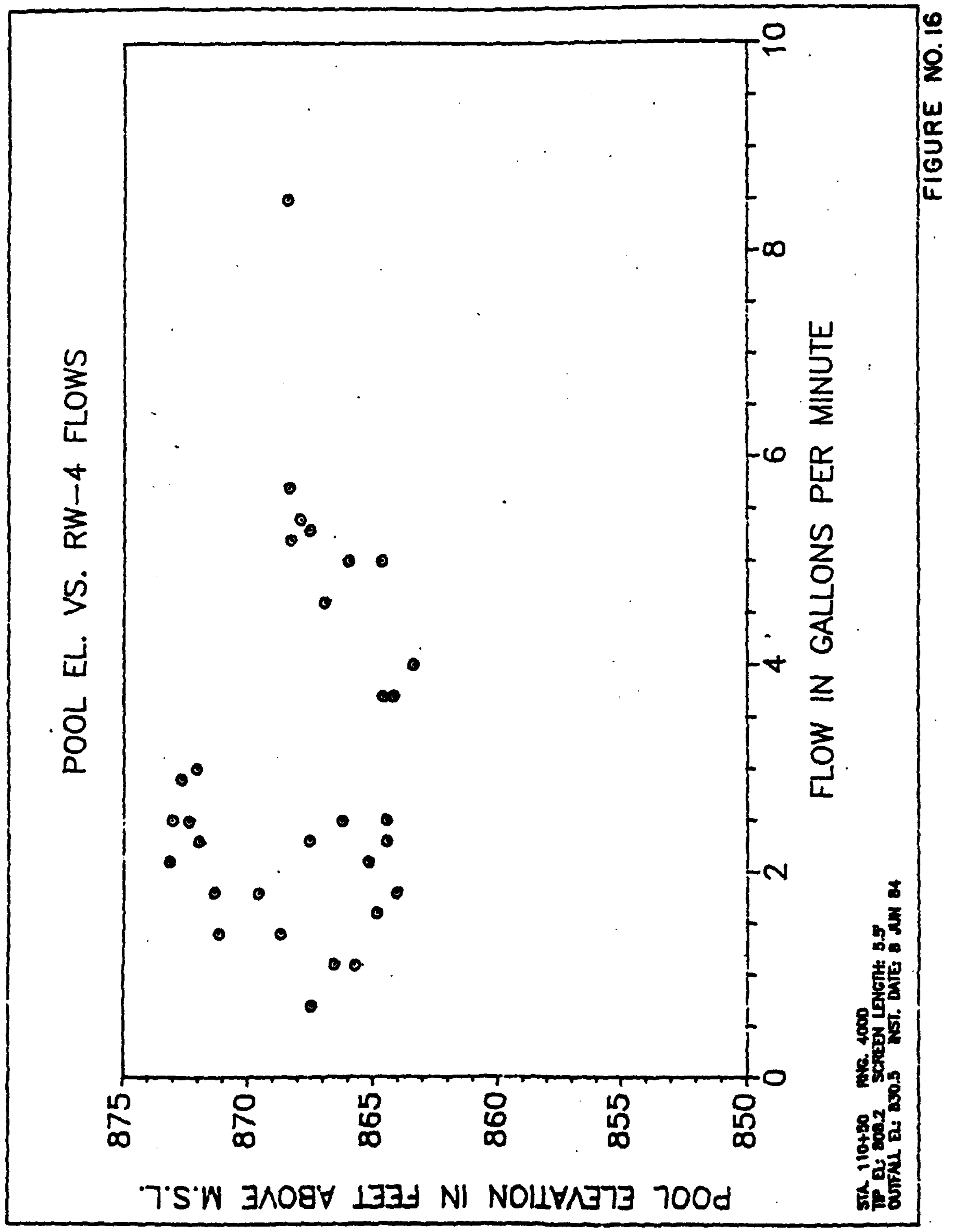

POOL EL. VS. RW-4 FLOWS
POOL ELEVATION IN FEET ABOVE M.S.L.
FLOW IN GALLONS PER MINUTE
875
870
865
860
855
850
0
2
4
6
8
10
FIGURE NO.16

Figure No. 16

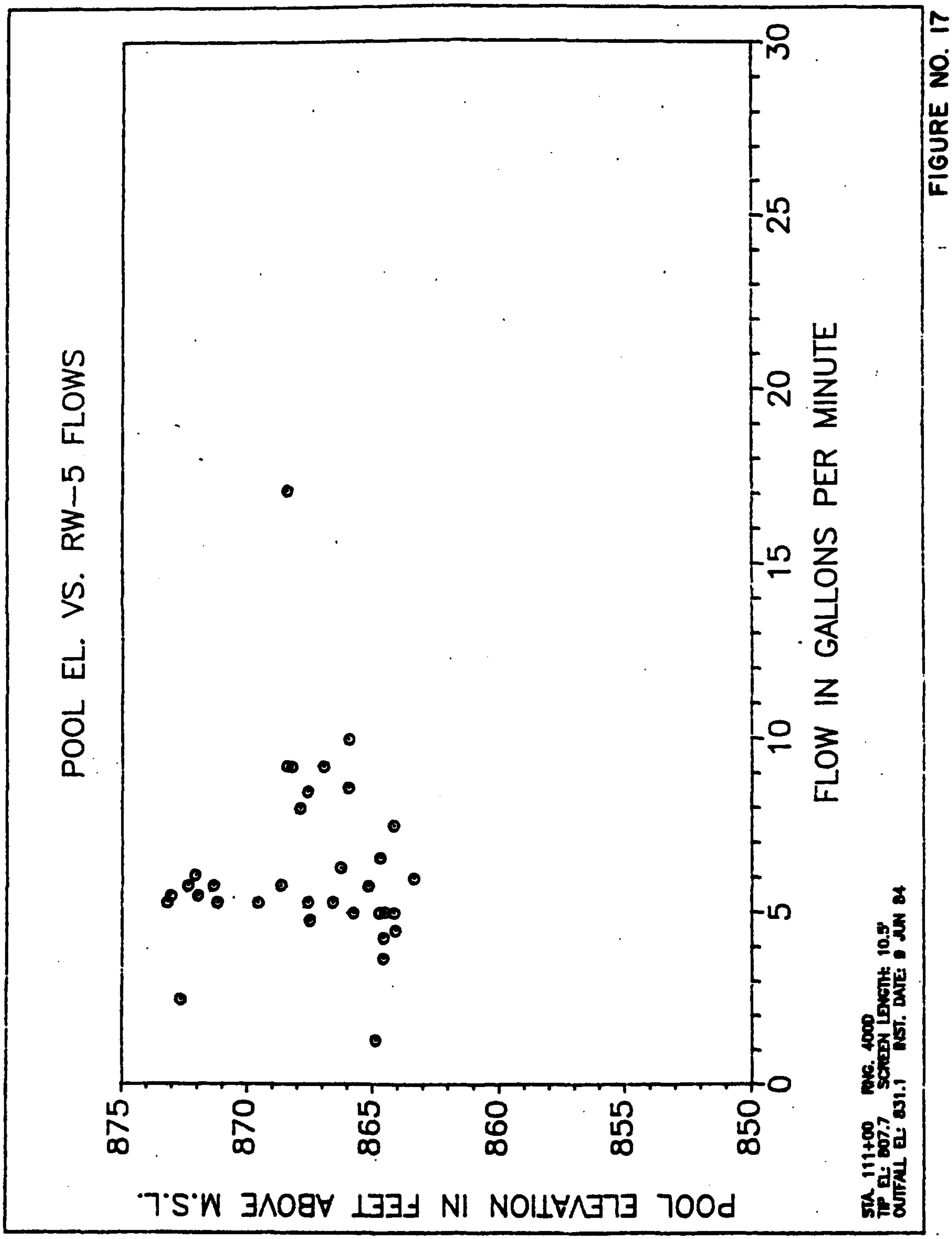
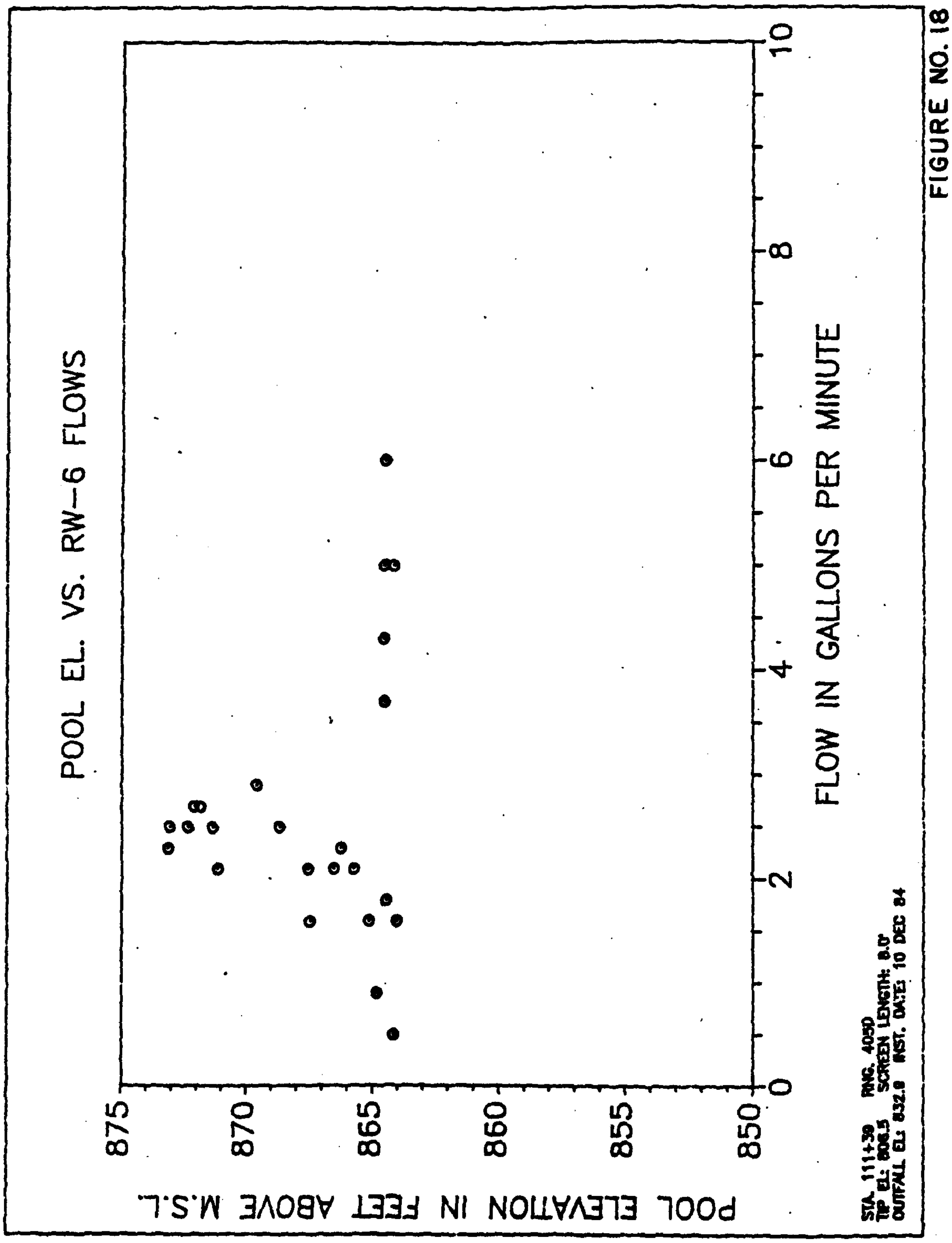

POOL EL. VS. RW-5 FLOWS
POOL ELEVATION IN FEET ABOVE M.S.L.
875
870
865
860
855
850
0
5
10
15
20
25
30
FLOW IN GALLONS PER MINUTE
STA. 111+00 RNG. 4000
TIP EL: 807.7 SCREEN LENGTH: 10.5'
OUTFALL EL: 831.1 INST. DATE: 9 JUN 84
FIGURE NO. 17

Figure No. 17



POOL EL. VS. RW-6 FLOWS
POOL ELEVATION IN FEET ABOVE M.S.L.
875
870
865
860
855
850
0
2
4
6
8
10
FLOW IN GALLONS PER MINUTE
STA. 111+30 RNG. 405D
TIP EL: 806.5 SCREEN LENGTH: 5.0'
OUTFALL EL: 832.8 INST. DATE: 10 DEC 84
FIGURE NO. 18

Figure No. 18

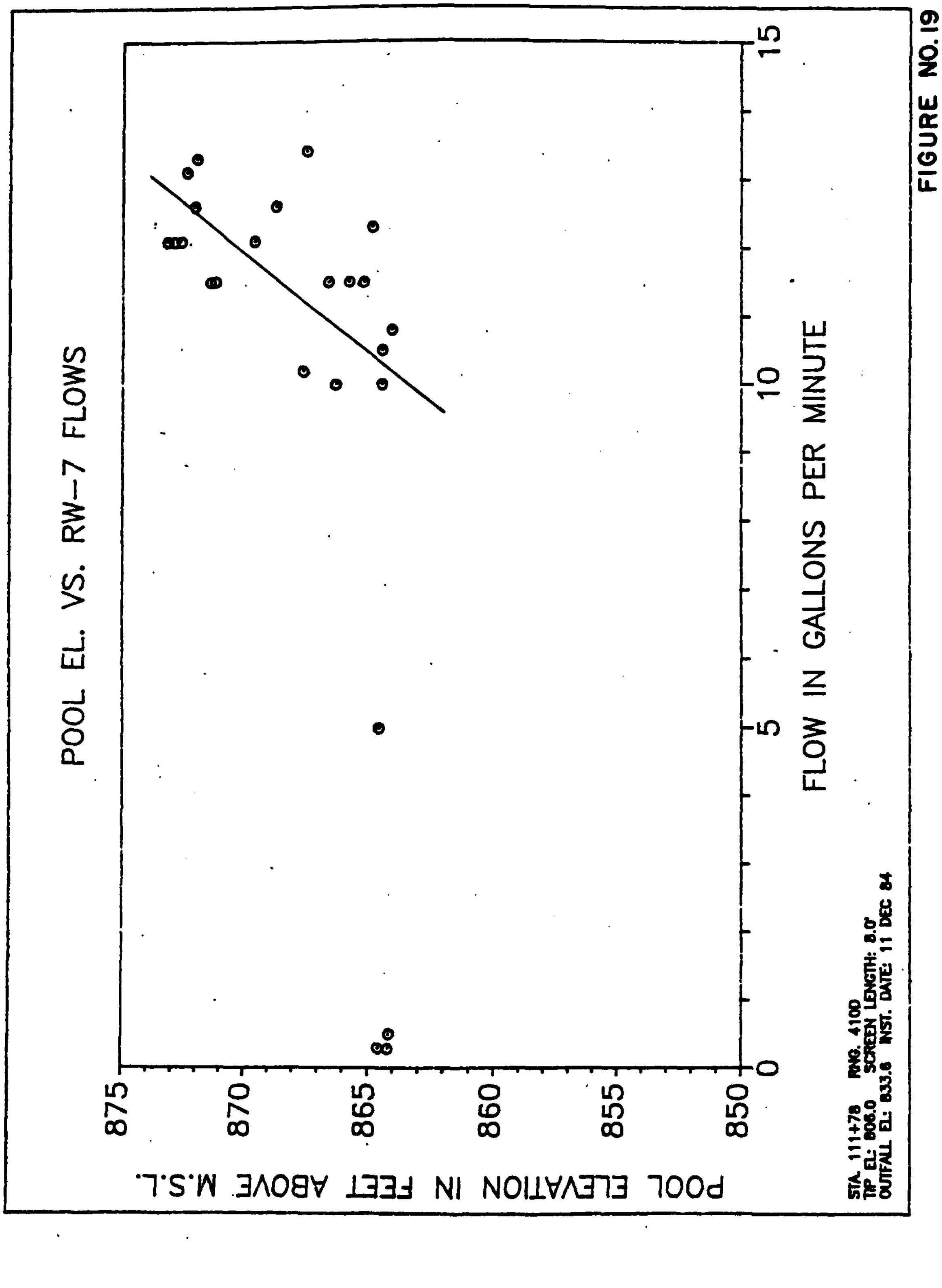
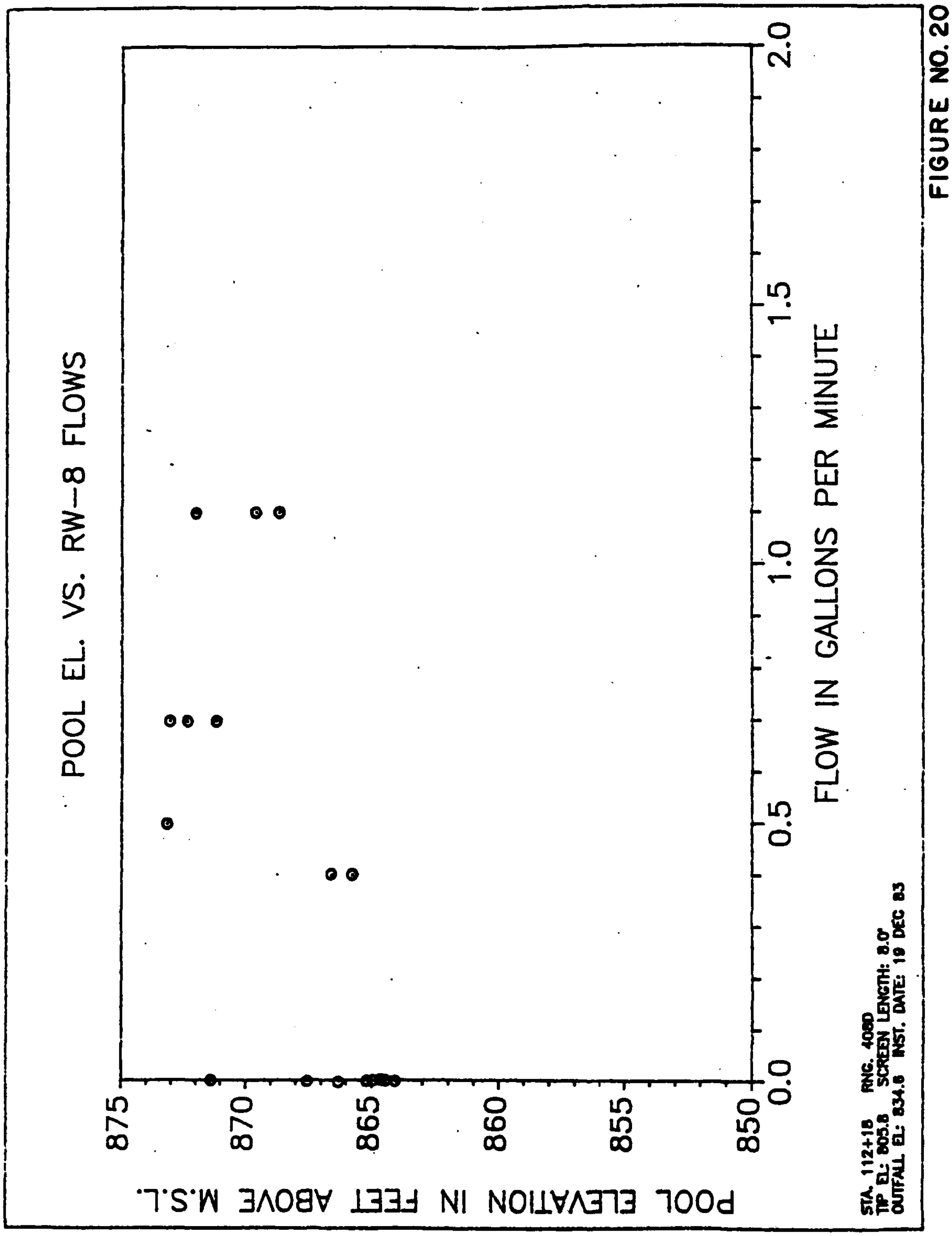

POOL EL. VS. RW-7 FLOWS
FIGURE NO.19
FLOW IN GALLONS PER MINUTE
POOL ELEVATION IN FEET ABOVE M.S.L.
875
870
865
860
855
850
0
5
10
15
STA. 111+78 RNG. 410D
TIP EL: 806.0 SCREEN LENGTH: 8.0'
OUTFALL EL: 833.6 INST. DATE: 11 DEC 84

Figure No. 19



POOL EL. VS. RW-8 FLOWS
POOL ELEVATION IN FEET ABOVE M.S.L.
875
870
865
860
855
850
0.0
0.5
1.0
1.5
2.0
FLOW IN GALLONS PER MINUTE
STA. 112+18 RNG. 4080
TIP EL: 805.8 SCREEN LENGTH: 8.0'
OUTFALL EL: 834.6 INST. DATE: 19 DEC 83
FIGURE NO. 20

Figure No. 20

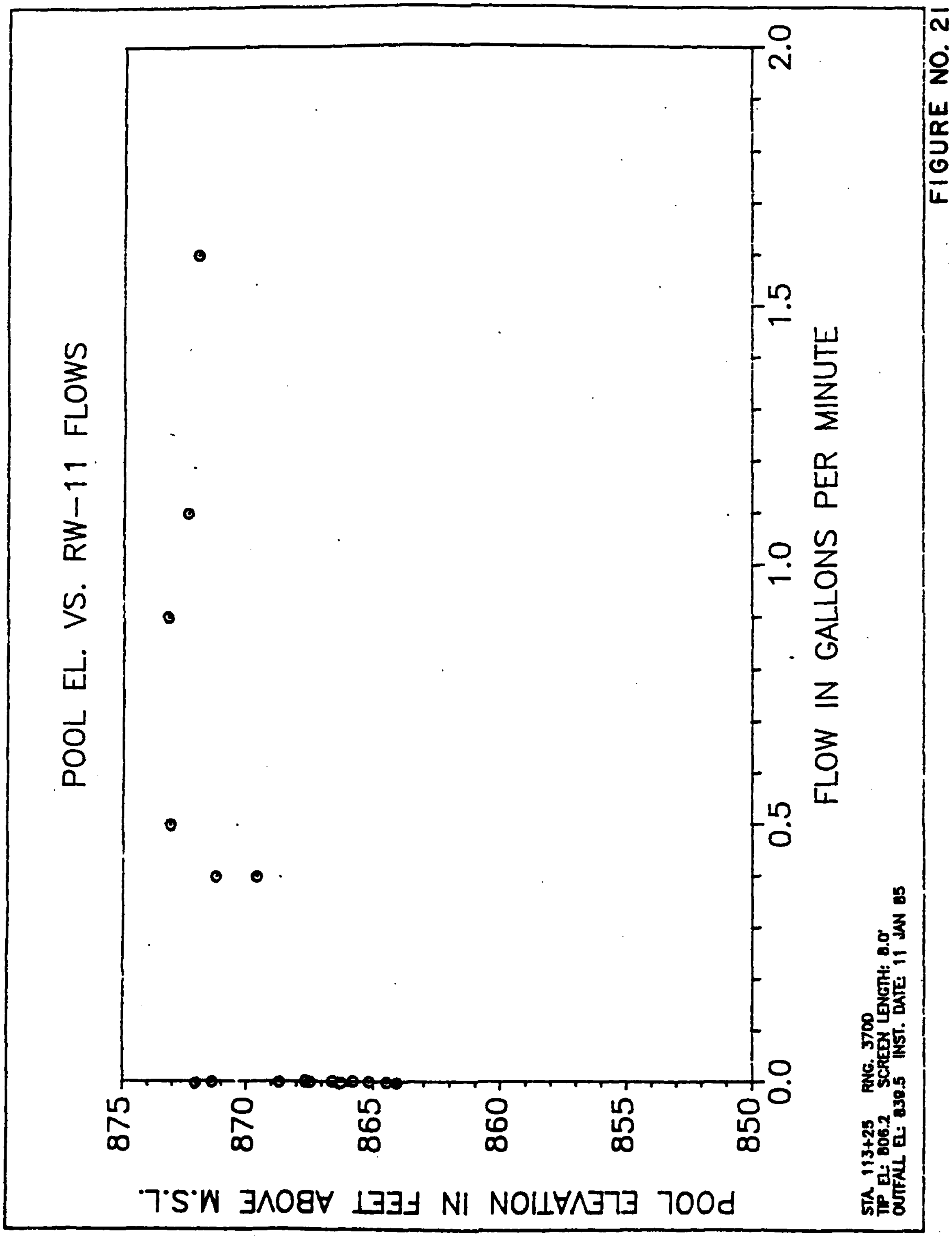

POOL EL. VS. RW-11 FLOWS
POOL ELEVATION IN FEET ABOVE M.S.L.
875
870
865
860
855
850
0.0
0.5
1.0
1.5
2.0
FLOW IN GALLONS PER MINUTE
STA. 113+25 RNG. 3700
TIP EL: 806.2 SCREEN LENGTH: 8.0'
OUTFALL EL: 839.5 INST. DATE: 11 JAN 85
FIGURE NO. 21

Figure No. 21

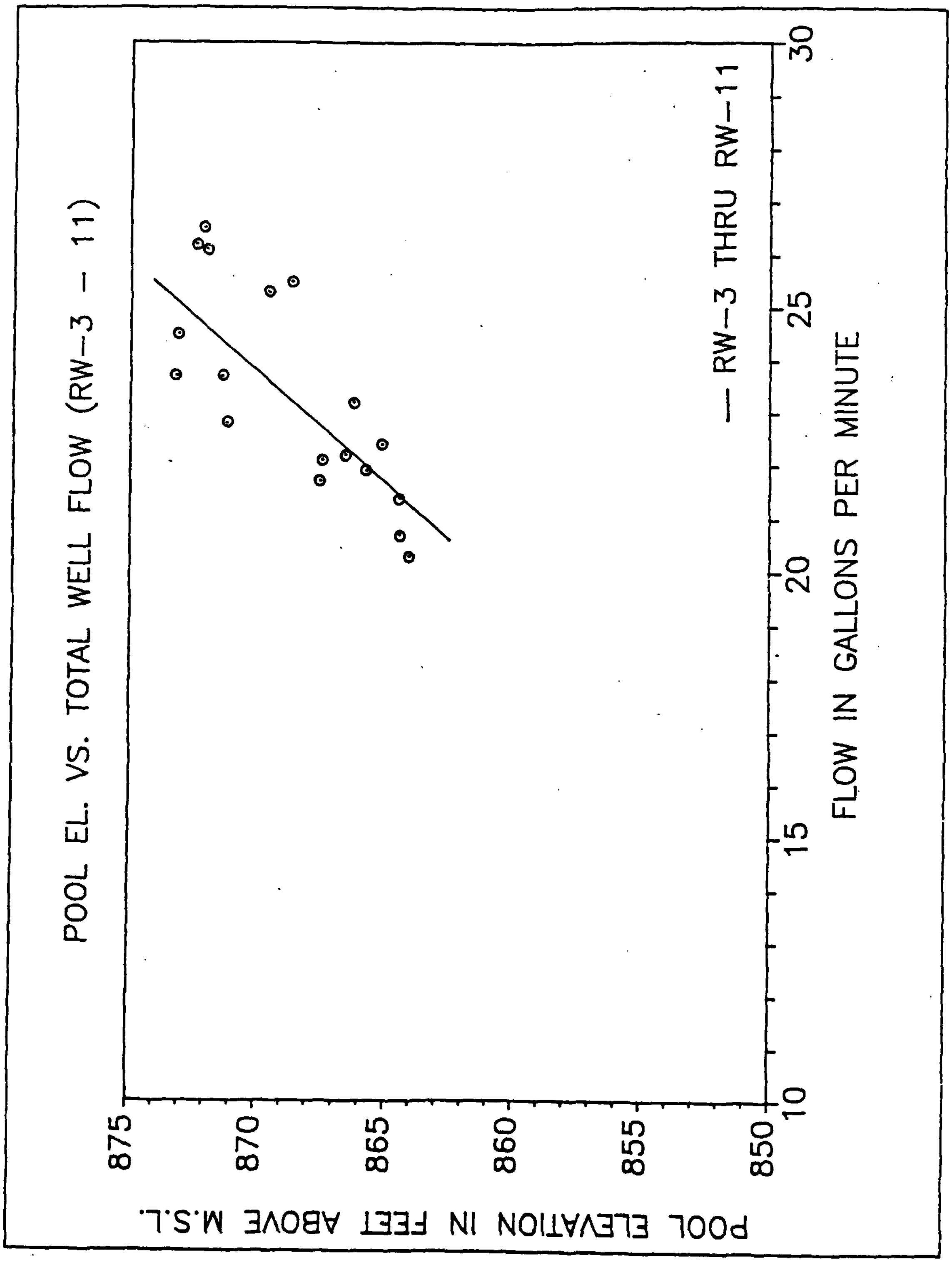
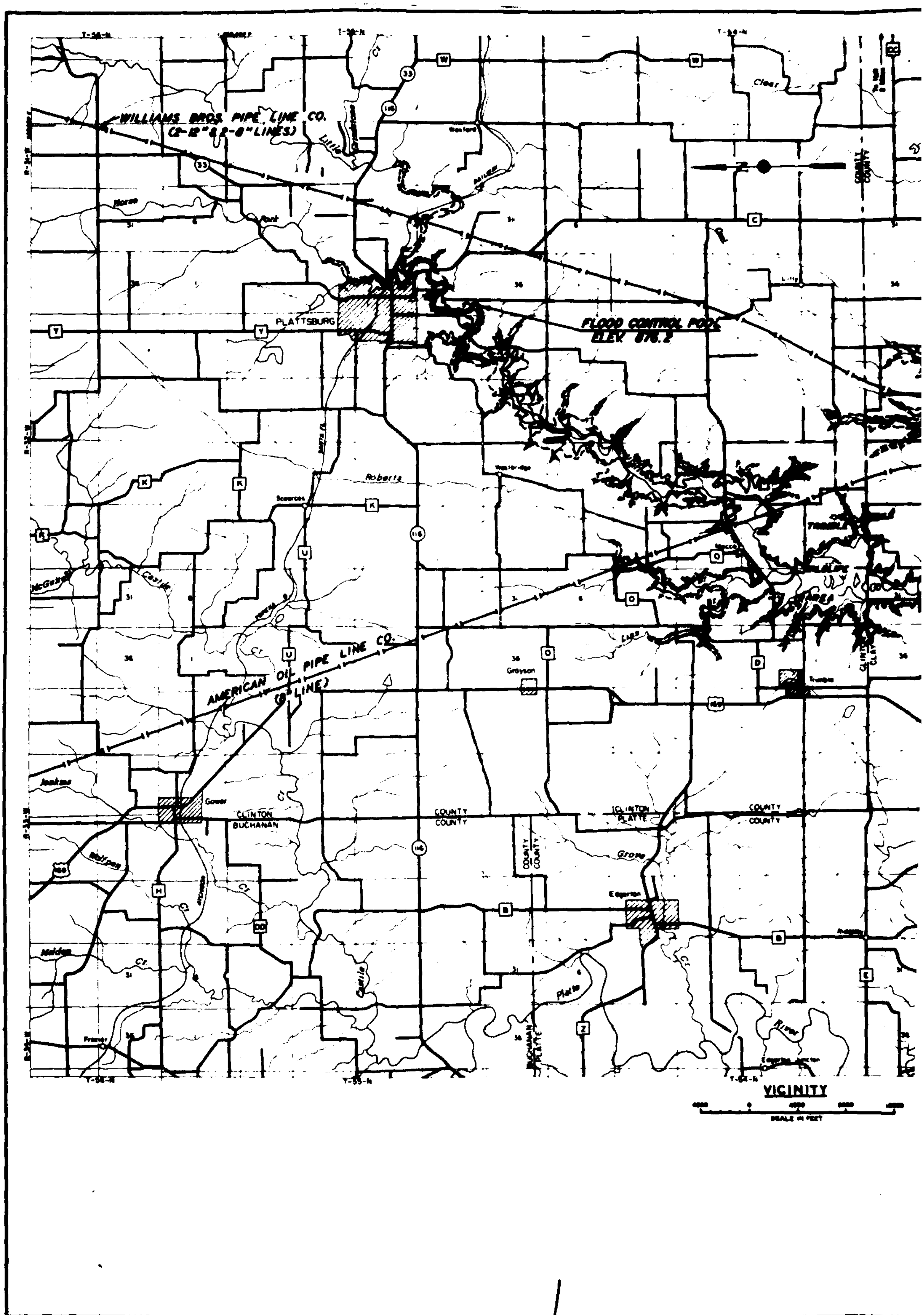


FIGURE NO. 22

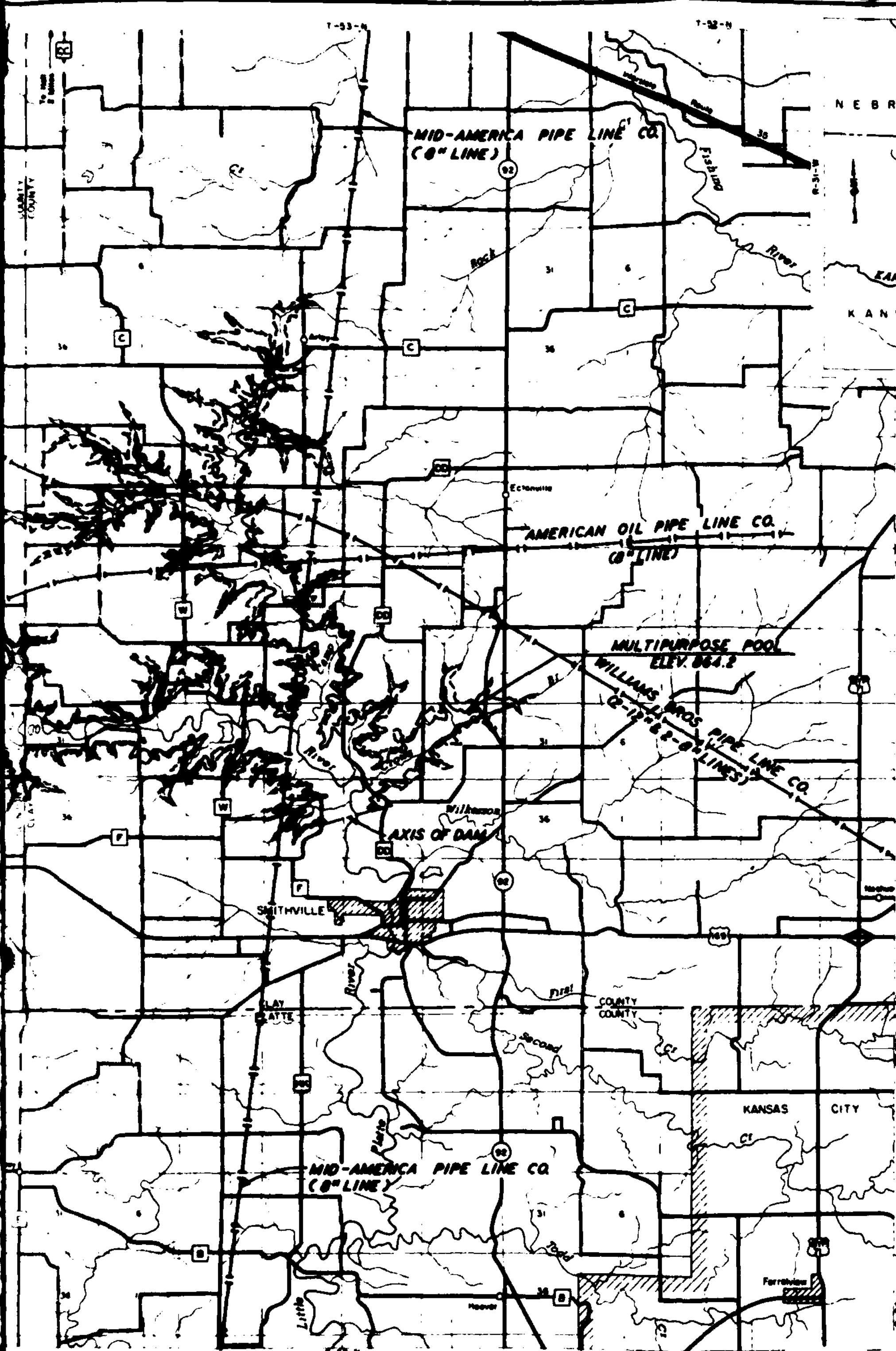
Figure No. 22

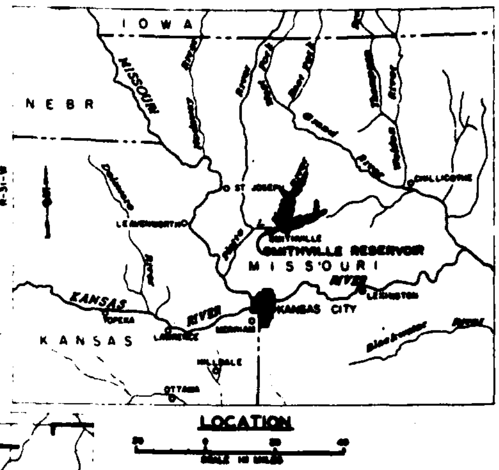
DRAWINGS

DRAWINGS



WILLIAMS BROS. PIPE LINE CO.
(2-12" & 2-8" LINES)
FLOOD CONTROL POOL
PLATTSBURG
Roberts
Scearces
AMERICAN OIL PIPE LINE CO.
(8" LINE)
Grayson
Trimble
Gower
Jenkins
CLINTON
BUCHANAN
COUNTY
COUNTY
CLINTON
PLATTE
Grove
Edgerton
Castile
Platte
River
VICINITY
SCALE IN FEET





SIGNATURES AFFIXED BELOW INDICATE OFFICIAL RECOMMENDATION AND APPROVAL OF DRAWINGS IN THIS SET AS INDEXED ON THIS SHEET

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT

LOCATION VICINITY MAP

Designed by:	J.D.H.
Drawn by:	R.A.C.
Checked by:	J.D.H.
Submitted by:	F.C.M.

Scale: AS SHOWN

MARCH 1967

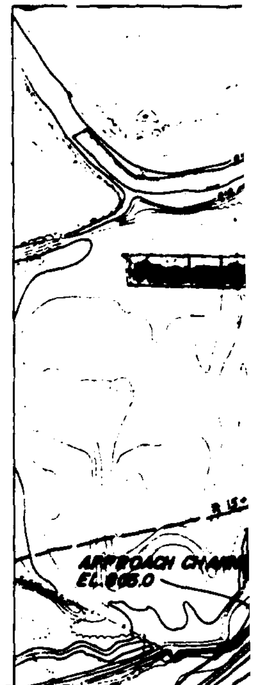
File No. RP-3-1751

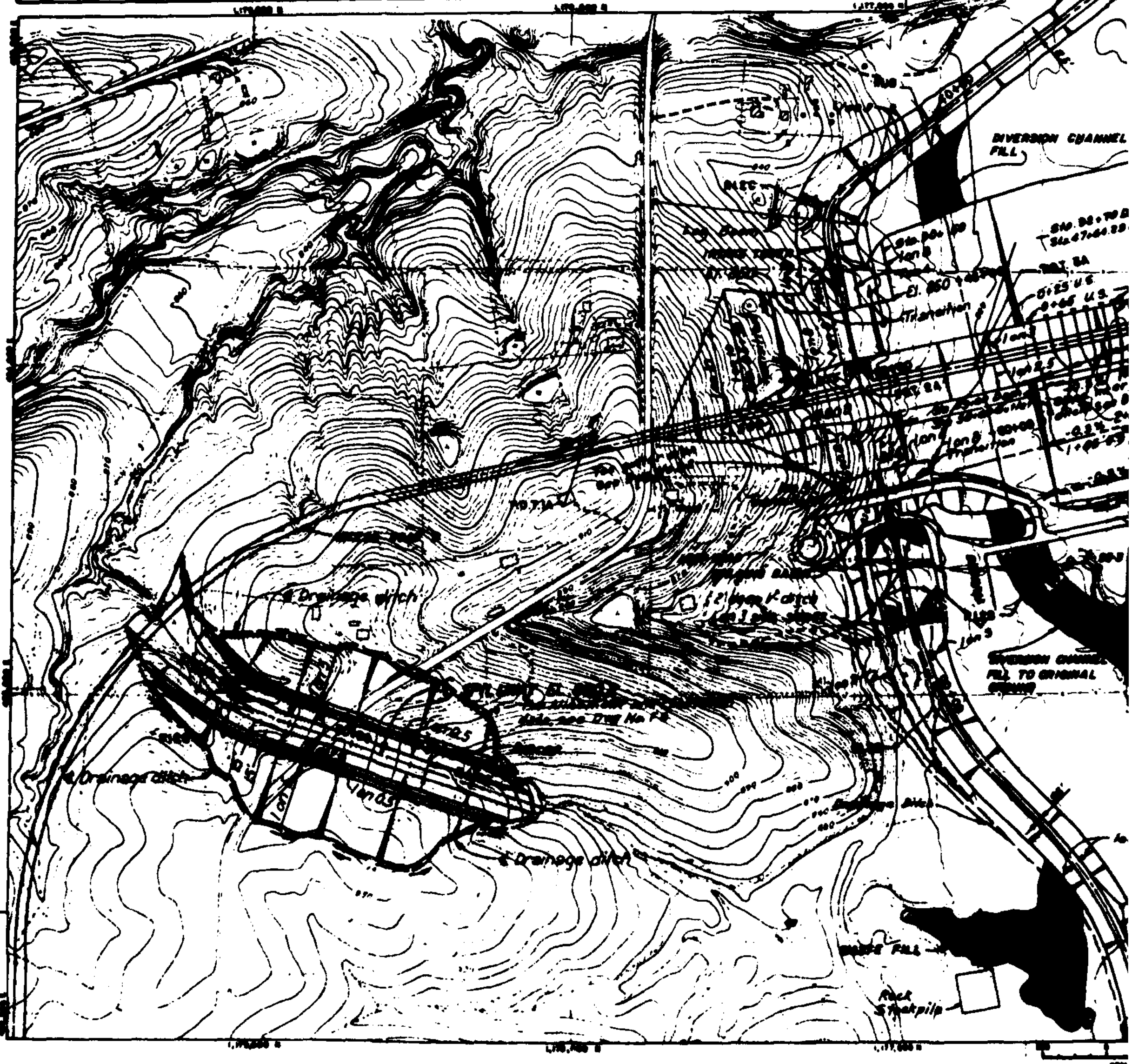
Revisions			
Symbol	Descriptions	Date	Approved

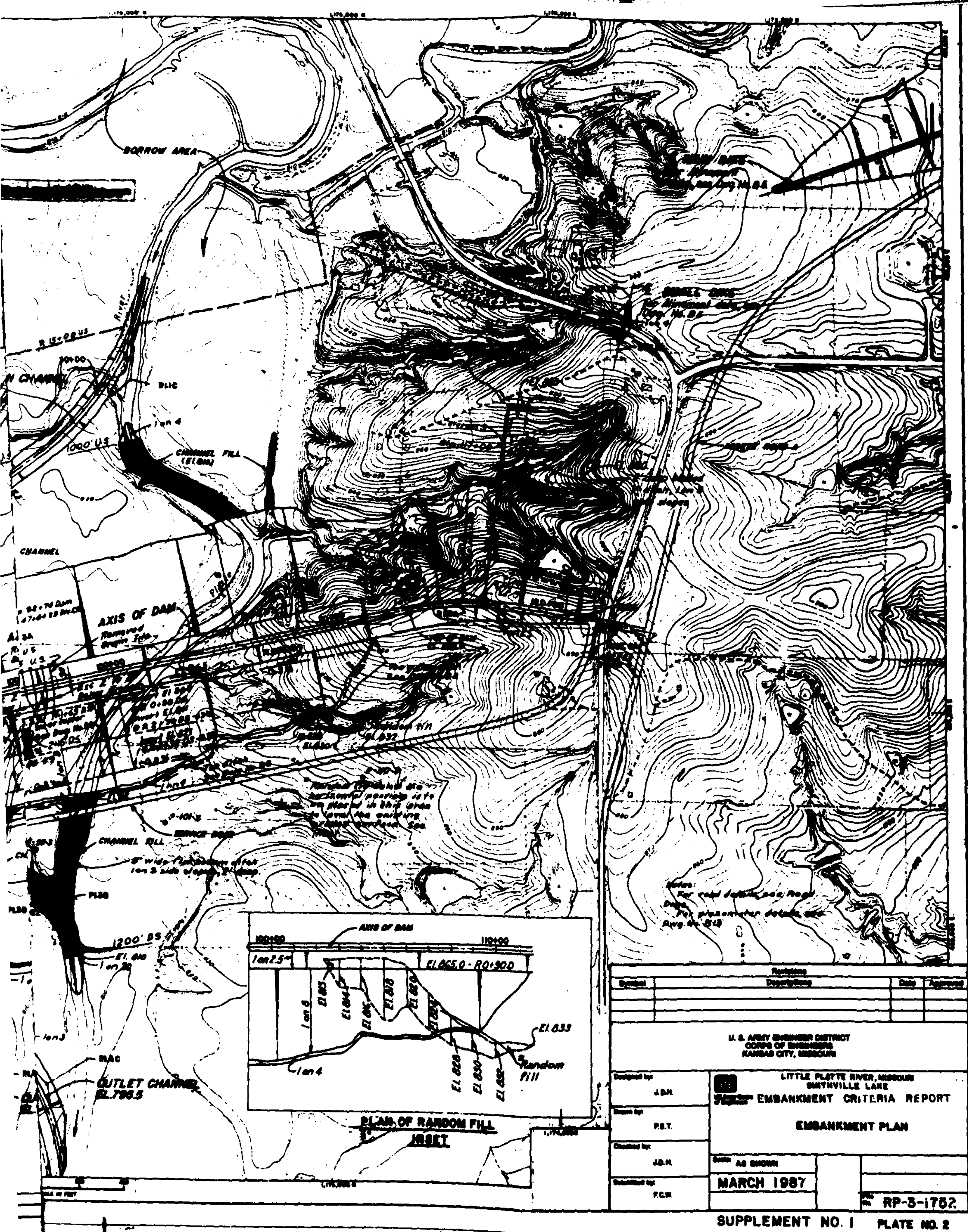
SUPPLEMENT NO 1 PLATE NO. 1

ALINEMENT DATA

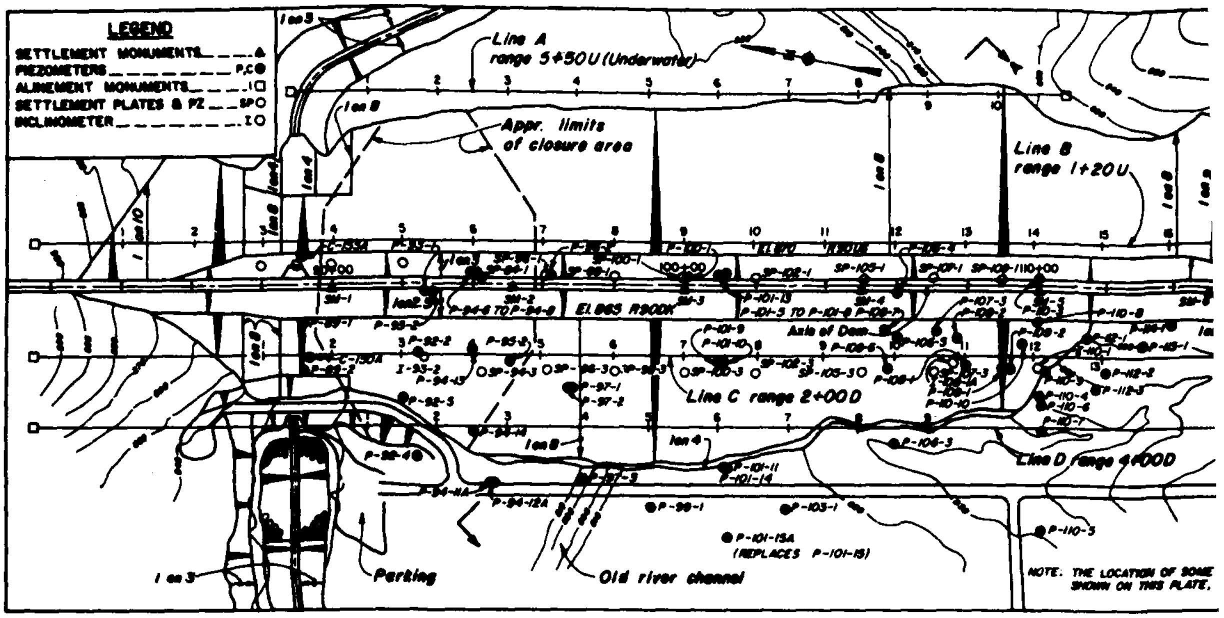
P.I.	POT	LATITUDE	DEPARTURE	AZIMUTH	DISTANCE	STATION	P.C. STA.	P.T. STA.	Δ	D	T	R	L
AXIS DAM													
	1A	1,177,372.67	[illegible]			80+00							
				347°30'	900.00'								
	2A	1,177,094.00	[illegible].00			89+00 = Station 50+00 Outlet Works							
				347°30'	370.00'								
	3A	1,176,732.77	483,995.06			92+70 = Station 47+64.29 Diversion Channel							
				347°30'	2281.65'								
4A		1,174,506.38	484,[illegible]				114+00.09	117+00.09	20° Rt.	6°40'	151.54'	859.41'	300.00'
				7°30'	640.00'								
5A		1,173,870.70	484,[illegible]				119+68.47	123+68.47	4° Rt.	1°	200.06	5729.58	400.00'
				11°30'	800.00'								
	6A	1,173,576.77	484,945.57			124+[illegible]							
DIVERSION CHANNEL													
				90°									
1C		1,175,942.23	[illegible]				24+15.48	33+05.48	44°30' Rt.	5°	468.81'	1145.92'	890.00'
				142°30'	1201.75'								
1B		1,176,[illegible]	484,723.70				39+16.75	41+33.42	65° Lt.	30°	121.67'	190.98'	216.67'
				77°30'	752.54'								
	3A	1,176,732.77	483,995.06			47+64.29 = Station 92+70 Dam Axis							
				77°30'	700.00'								
2B		1,176,[illegible]	483,[illegible].67				53+00.18	56+10.18	46°30' Lt.	15°	164.11'	381.97'	310.00'
				31°00'	410.00'								
3B		[illegible]	483,100.[illegible]				56+91.96	60+01.96	46°30' Rt.	15°	164.11'	381.97'	310.00'
				77°30'									
OUTLET WORKS													
				90°									
1C		1,173,912.28	[illegible]				24+15.48	33+05.48	44°30' Rt.	5°	468.81'	1145.92'	890.00'
				142°30'	1610.00'								
2C		1,177,[illegible]	[illegible]				43+25.00	45+41.67	65° Lt.	30°	121.67'	190.98'	216.67'
				77°30'	580.00'								
	2A	1,177,094.00	[illegible].00			50+00 = Station 89+00 Dam Axis							
				77°30'	1020.73'								
3C		1,176,[illegible]	482,[illegible].47				57+50	62+70	32° Lt.	6°09'	266.37'	931.03'	520.00'
				63°30'	1010.21'								
4C		1,176,[illegible]	[illegible]				65+34	74+04	51° Rt.	6°	485.48'	954.93'	850.00'
				96°30'									







SUPPLEMENT NO. 1 PLATE NO. 2



PLAN OF OBSERVATION DEVICES

SCALE IN FEET

ALINEMENT MONUMENTS		
NUMBER	STATION	ORIGINAL TOP ELEVATION
ALINEMENT LINE "A" IS LOCATED 5+50 US AND IS PERMANENTLY UNDERWATER		
A-1	91+00	823.93
A-2	93+00	822.10
A-3	95+00	821.71
A-4	97+00	819.75
A-5	99+00	820.06
A-6	101+00	818.70
A-7	103+00	816.39
A-8	105+00	818.74
A-9	107+00	816.32
A-10	109+00	815.79
ALINEMENT LINE "B" IS LOCATED 1+20 US		
B-1	84+00	865.51
B-2	86+00	861.40
B-3	88+00	867.25
B-4	90+00	867.22
B-5	92+00	867.23
B-6	94+00	867.23
B-7	96+00	867.29
B-8	98+00	866.77
B-9	100+00	866.70
B-10	102+00	866.96
B-11	104+00	867.06
B-12	106+00	867.19
B-13	108+00	867.28
B-14	110+00	867.32
B-15	112+00	867.30
B-16	114+00	867.32
B-17	116+00	875.44
B-18	118+00	877.90
B-19	120+00	872.58
ALINEMENT LINE "C" IS LOCATED 2+00 DS		
C-1	88+00	853.64
C-2	90+00	852.23
C-3	92+00	852.03
C-4	94+00	851.99
C-5	96+00	852.15
C-6	98+00	852.18
C-7	100+00	852.30
C-8	102+00	852.33
C-9	104+00	852.34
C-10	106+00	852.35
C-11	108+00	852.39
C-12	110+00	852.41
C-13	112+00	866.49
ALINEMENT LINE "D" IS LOCATED 4+00 DS		
D-1	91+00	827.37
D-2	93+00	827.30
D-3	95+00	827.41
D-4	97+00	827.31
D-5	99+00	827.32
D-6	101+00	827.37
D-7	103+00	827.62
D-8	105+00	NO PLATE
D-9	107+00	827.15

TRILATERATION SURVEY

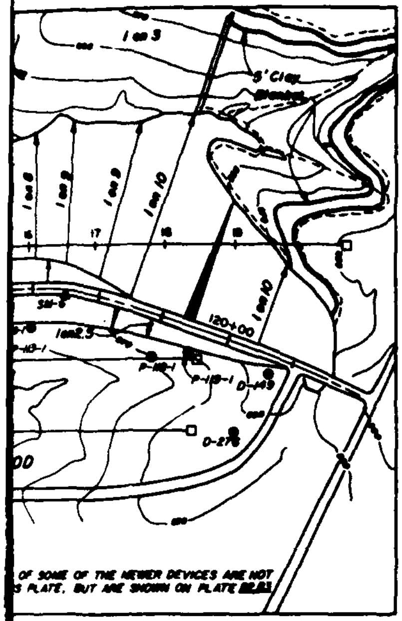
ALINEMENT LINE "B" MONUMENTS AND ONE POINT ON THE UPSTREAM EDGE OF THE INTAKE TOWER ROOF ARE USED FOR THE TRILATERATION SURVEY. B1 IS DESIGNATED A1, B2 IS DESIGNATED A2, ETC. B19 IS DESIGNATED A19. THE TOWER IS DESIGNATED AS A20.

INCLINOMETER SCHEDULE			
NUMBER	STATION	RANGE	BOTTOM ELEVATION
I-93-2	92+70	2+030	762.8
I-108-1A	108+03	2+34.50	782.9
I-110-1	110+00	2+400	781.1

CREST SETTLEMENT MONUMENTS			
NUMBER	STATION	RANGE	ORIGINAL TOP ELEVATION
SM-1	90+00	0+140	894.93
SM-2	95+00	0+140	894.94
SM-3	100+00	0+140	894.91
SM-4	105+00	0+140	894.94
SM-5	110+00	0+140	895.00
SM-6	115+00	0+140	894.95

SETTLEMENT PLATES W/PIEZOMETERS				
NUMBER	STATION	RANGE	ORIGINAL PLATE ELEVATION	TOP ELEVATION
SP-94-1	94+20	0+20U	813.67	895.01
SP-94-3	94+20	2+30D	814.11	846.45
SP-96-1	96+00	0+20U	813.54	894.66
SP-96-3	96+00	2+30D	813.34	845.62
SP-98-1	98+00	0+20U	813.27	893.81
SP-98-3	98+00	2+30D	815.15	845.85
SP-100-1	100+00	0+20U	811.43	893.92
SP-100-3	100+00	2+30D	813.87	847.26
SP-102-1	102+00	0+20U	811.31	893.85
SP-102-3	102+00	2+30D	811.24	846.62
SP-105-1	105+00	0+20U	814.20	894.46
SP-105-3	105+00	2+30D	816.50	847.33
SP-107-1	107+00	0+20U	839.99	895.00
SP-107-3	107+00	2+30D	822.30	847.05
SP-109-1	109+00	0+20U	841.91	894.73

PIEZO[illegible]			
NUMBER	STATION	RANGE	EL[illegible]
P-89-1	89+25	1+00D	
P-89-2	89+25	2+30D	
C-130A	89+30.5	2+10D	
P-92-2	92+50	2+00D	
P-92-4	92+60	4+00D	
P-92-5	92+00	3+23D	
P-93-1	92+64	0+15D	
P-93-2	92+75	0+15D	
P-94-6	94+00	0+30U	
P-94-7	94+10	0+20U	
P-94-8	94+10	0+30U	
P-94-11A	94+36	5+62D	
P-94-12A	94+66	5+62D	
P-94-13	94+04	1+81D	
P-94-14	94+00	4+80D	
P-95-2	95+05	2+20D	
P-96-2	96+20.5	0+20U	
P-97-1	96+70	3+00D	
P-97-2	96+90	3+00D	
P-97-3	97+00	5+60D	
P-99-1	99+00	6+38D	
P-100-1	100+05	0+20U	
P-101-5	100+95	0+20U	
P-101-6	100+95	0+30U	
P-101-7	101+05	0+20U	
P-101-8	101+05	0+30U	
P-101-9	100+95	2+20D	
P-101-10	101+05	2+20D	
P-101-11	101+00	3+30D	
P-101-13	101+00	0+25U	
P-101-14	101+10	3+20D	
P-101-15A	101+00	7+23D	
P-103-1	103+00	6+36D	
P-106-3	106+00	4+65D	
P-106-4	106+02	0+15D	
P-106-5	106+00	1+30D	
P-106-6	106+56	2+40D	
P-106-7	105+50	1+30D	
P-107-3	107+00	1+36D	
P-108-1	108+03	2+34D	
P-108-2	107+60	1+20D	
P-108-3	107+30	3+20D	
P-109-1	109+00	2+40D	
P-109-2	109+10	2+40D	
P-110-3	110+00	0+20U	
P-110-4	110+00	3+20D	
P-110-5	110+00	7+00D	
P-110-6	110+00	3+40D	
P-110-7	110+00	4+23D	
P-110-8	110+00	1+09D	
P-110-9	110+10	2+40D	
P-110-10	109+47.2	1+35.50	
P-110-11	110+00	[illegible]	
P-112-1	111+80	1+20D	
P-112-2	112+00	2+60D	
P-112-3	111+30	3+00D	
P-112-4	111+30	3+65D	
P-112-5	111+50	5+00D	
P-113-1	113+00	1+77D	
P-113-2	112+75	3+78D	



PIEZOMETER SCHEDULE (FOR DAM)

NUMBER	STATION	RANGE	TIP ELEVATION	MATERIAL	TOP ELEVATION	DATE INSTALLED	GROUND ELEVATION
P-114-1	114+00	1+100	817.1	SANDY CLAY	869.08	10-2-74	865.1
P-114-2	113+50	2+200	810.9	CLY GRV/GRV	864.6	10-29-84	861.8
P-114-3	114+50	3+600	807.0	GRVY SAND	858.0	11-13-84	862.1
P-114-4	113+50	4+900	806.7	GRAVEL	851.8	11-7-84	848.6
P-118-1	118+00	1+000	807.00	GRVY CLY SD	872.33	11-8-83	868.8
P-119-1	119+00	0+750	824.0	CLY SDY GRAVEL	880.67	10-3-74	877.0
D-147(E)			875.2	SIY CLY SD	902.11	4-12-71	898.3
D-147(W)			832.5	SIY GRVY SD	902.11	4-12-71	898.3
D-148			838.2	LEAN CL/SD/SILT	886.4	4-16-71	882.0
D-149(N)	120+15	0+200	866.7	CLY GRV	868.39	4-6-71	862.8
D-278	120+00	3+000	808.86	SIY LEAN CL	869.46	6-5-74	867.7
DC-513			810.0	CL SDY GRV	854.73	9-21-83	851.6
D-514			817.66	CLY SAND	846.03	10-13-83	840.96
DC-514-A			809.66	CLY GRVY SD	846.01	10-14-83	841.36
D-515			807.6	SIY GRV SD	879.67	9-29-83	877.1
D-516			812.7	SIY GRV SD	896.37	10-3-83	892.9
DC-517			812.9	SIY GRV SD	883.98	10-7-83	880.0
D-518			823.3	SDY CLY GRV	869.89	10-20-83	866.3
D-519A			820.95	SDY LEAN CL	842.6	4-3-84	838.86
A-520			812.13	SDY CL	831.56	2-9-84	828.83
A-521			810.25	SIY SD	834.30	2-10-84	830.25
D-521A			808.99	SIY SAND	833.26	4-3-84	829.99
AD-522			812.16	CLY SAND	834.30	3-28-84	834.09
AD-523			793.36	SDY CLY	821.86	3-27-84	821.86
AD-524			818.99	LEAN CLAY	827.31	3-29-84	826.99

*MATERIAL DESCRIPTION FROM FIELD VISUAL CLASSIFICATION

PIEZOMETER SCHEDULE (FOR DAM)

RANGE	TIP ELEVATION	MATERIAL	TOP ELEVATION	DATE INSTALLED	GROUND ELEVATION
000	805.7	SAND DRAIN	866.26	9-20-76	864.5
300	797.1	SAND DRAIN	848.44	9-29-76	845.5
100	790.3	CHANUTE SHALE	852+00	9-8-76	849.7
000	775.0	QUIVIRA SHALE	864.33	9-8-76	850.4
800	786.7	WEA SHALE	819.37	10-22-76	817.7
230	791.41	SLY GRV SD	838.25	11-30-83	836.4
150	789.56	SD	868.62	12-6-83	865.1
150	829.23	SDY CL	898.29	12-2-83	894.7
30U	801.8	LEAN CLAY	891.84	4-20-74	887.3
20U	788.7	CLY GRVY SAND	894.01	4-20-74	891.5
30U	775.5	QUIVIRA SHALE	891.05	4-19-74	887.4
620	804.0	SILTY CLAY	819.29	8-18-75	813.5
620	791.0	LEAN CLAY	818.27	8-19-75	814.3
810	787.51	CLY SDY GRV	866.93	10-28-83	863.0
200	784.7	SIY GRV SD	826.46	11-1-83	823.7
200	791.0	*LEAN CLAY	850.20	3-26-75	848.2
[illegible]	823.0	*SND-LEAN CLAY	894.58	3-10-75	890.7
[illegible]	790.8	*SANDY CLAY	841.01	3-11-75	838.4
[illegible]	788.2	*LEAN CLAY	840.76	3-7-75	838.3
800	787.4	*CLAYEY SILT	817.52	2-19-81	813.4
380	788.3	*CLAYEY SILT	815.60	8-26-81	810.3
20U	748.1	WEA SHALE	889.84	4-1-75	881.8
20U	823.3	*SND-LEAN CLAY	888.30	10-8-74	880.1
30U	790.0	LEAN CLAY	891.02	4-10-74	887.1
20U	788.2	*SANDY GRAVEL	894.97	4-17-74	891.5
30U	780.1	WEA SHALE	891.93	4-19-74	887.1
200	798.8	LEAN CLAY	851.72	3-28-75	849.5
200	790.2	*LEAN CLAY	852.10	3-27-75	848.8
200	798.4	LEAN CLAY	819.69	3-8-76	813.0
23U	738.1	WEA SHALE	898.60	4-12-74	889.3
200	776.2	QUIVIRA SHALE	820.17	9-9-76	815.4
230	788.3	*CLAYEY SILT	819.86	2-18-81	814.2
380	794.0	*LEAN CLAY	819.46	2-19-81	814.0
380	809.9	CLAYEY SAND	832.89	10-4-79	828.1
150	807.67	SIY GRV SD	897.68	12-9-83	894.9
800	808.7	CLAY W/GRV	889.84	3-17-84	887.4
400	802.8	SDY SIY CL	848.7	3-26-84	846.4
300	798.0	CHANUTE SH	852.27	3-21-84	850.0
360	808.7	SDY CLAY	891.36	3-19-84	889.3
340	801.4	MUNCIE CRK SH	850.75	4-20-84	847.4
00	810.2	SDY CL	889.73	3-19-84	887.7
300	811.5	SIY SAND	868.9	11-8-84	865.3
400	809.9	SIY CL	851.07	3-9-84	848.9
400	789.8	CHANUTE SH	851.49	3-19-84	848.8
20U	813.4	CLY SDY GRAVEL	886.18	9-7-74	882.0
200	813.3	SANDY CLAY	863.12	10-2-74	859.1
900	822.0	SANDY CLAY	865.64	8-9-74	861.8
400	806.1	SDY CL GRV	842.1	10-30-83	838.1
230	809.5	CL GRV	856.44	11-22-83	852.6
990	811.00	CLY SDY GRV	866.80	11-17-83	863.0
400	809.45	LS COBBLES	849.36	3-2-84	846.78
0.30	809.66	SA LS SH SEAM	852.34	4-2-84	849.65
900	807.4	SDY GRV	866.2	11-29-84	863.0
200	813.6	SIY SDY GRV	880.39	3-11-84	877.8
800	802.75	SA LS SH SEAM	854.46	9-7-84	851.75
900	808.1	GRVS W/CLY SI	849.10	9-6-84	846.09
050	811.7	GRVY SAND	841.0	11-1-84	838.0
000	806.9	CLY SDY GRAVEL	848.4	11-8-84	845.3
70	818.4	SDY GRV	900.63	9-7-84	897.6
80	807.5	SDY GRV	853.9	10-31-84	849.2

PIEZOMETER SCHEDULE (FOR MAIN DIKE)

NUMBER	STATION	RANGE	TIP ELEVATION	MATERIAL	TOP ELEVATION	DATE INSTALLED	GROUND ELEVATION
P-18-1	19+00	1+400	831.5	CLY SIY SD	862.57	11-19-84	859.1
P-20-1	20+00	2+00U	840.0	SANDY CLAY	881.05	8-21-75	878.0
P-20-2	19+95	0+10U	840.1	SANDY CLAY	887.86	8-27-75	885.0
P-20-3	20+05	0+10U	821.7	ISLAND CREEK SH	867.90	9-2-75	865.0
P-20-4	20+00	2+150	835.0	*SANDY CLAY	863.30	8-21-75	860.4
P-22-1	22+00	0+800	829.6	CLY SD	870.99	11-21-84	867.8
P-22-2	22+00	2+000	828.4	SDY SI/CLY SD	860.46	11-15-84	856.7

*MATERIAL DESCRIPTION FROM FIELD PERMEABILITY TESTS

RELIEF WELL INSTALLATION SCHEDULE RW-1 THRU RW-13

RELIEF WELL NO.	STATION	RANGE	APPROX. DRILLING DEPTH (FT.)	APPROX. BOTTOM OF HOLE (ELEV.)	BOTTOM OF WELL SCREEN APPROX. (ELEV.)	SCREEN LENGTH (FT.)	GRAVEL PACK (FT.)	BENTONITE BALL SEAL (FT.)	GROUT BACKFILL (FT.)	PVC RISER TO BOX OF TEE (FT.)	TOP OF PVC RISER (ELEV.)	OUTFLOW AT TEE (ELEV.)	DATE INSTALLED
1	109+04	3+470	30.0	802.8	804.8	10.0	16.7	3.0	10.3	17.4	833.9	831.8	6-6-84
2	109+60	3+500	38.0	796.6	806.6	10.5	23.5	0.0	15.5	16.9	837.07	833.8	5-2-84
3	110+00	4+000	34.0	802.8	805.3	8.0	16.4	1.8	12.8	16.7	838.3	830.5	12-6-84
4	110+50	4+000	30.1	803.9	808.2	5.5	16.3	3.7	10.1	16.8	836.5	830.4	6-8-84
5	111+00	4+000	29.9	805.1	807.7	10.5	17.7	2.7	9.3	12.9	838.2	831.1	6-8-84
6	111+30	4+050	32.0	803.5	804.5	8.0	17.0	2.0	13.0	17.8	837.05	832.9	12-10-84
7	111+75	4+100	36.0	801.6	806.5	8.0	17.9	2.2	14.9	19.4	839.04	833.6	12-11-84
8	112+18	4+000	36.0	801.6	805.1	8.0	17.0	2.2	16.8	20.8	839.33	834.6	12-13-84
9	112+58	3+980	38.0	802.3	804.8	8.0	15.2	2.0	20.8	23.6	841.60	837.3	12-17-84
10	112+80	3+750	40.0	801.3	807.2	8.0	18.5	2.1	18.4	22.4	842.79	839.3	1-2-85
11	113+25	3+700	42.0	800.4	806.2	8.0	20.0	3.1	18.9	24.6	843.80	839.3	1-10-85
12	107+15	1+300	33.0	804.5	828.7 / 808.4	6.0 / 8.0	10.6 / 22.0	2.0	18.4	23.8 / 11.2	836.70	*	1-22-85
13	108+00	1+300	36.5	802.5	832.2 / 806.4	6.0 / 8.0	12.0 / 27.5	3.0	13.0	20.9 / 15.6	839.25	*	1-23-85

WHERE TWO VALUES ARE GIVEN, THE ONE BELOW THE LINE IS PLACED LOWER IN THE RELIEF WELL.
*RELIEF WELLS 12 AND 13 DISCHARGE INTO SAND BLANKET

PUMPED WELL INSTALLATION SCHEDULE

WELL NO.	STATION	RANGE	APPROX. DRILLING DEPTH (FT.)	APPROX. BOTTOM OF HOLE (ELEV.)	BOTTOM OF WELL SCREEN (ELEV.)	SCREEN LENGTH (FT.)	GRAVEL PACK (FT.)	BENTONITE BALL SEAL (FT.)	GROUT BACKFILL (FT.)	6" PVC RISER (FT.)	TOP OF 6" PVC RISER (ELEV.)	DATE INSTALLED
1	109+70	1+070	64.5	798.7	808.2	10.0	24.5	0.0	40.0	51.5	864.85	4-30-84
3	113+00	1+050	62.4	801.2	805.6	21.0	28.2	2.0	30.0	38.9	866.65	5-8-84
4	108+175	1+560	48.6	804.2	811.3	5.0	19.1	2.0	27.5	38.3	864.45	5-25-84

Revisions			
Symbol	Descriptions	Date	Approved

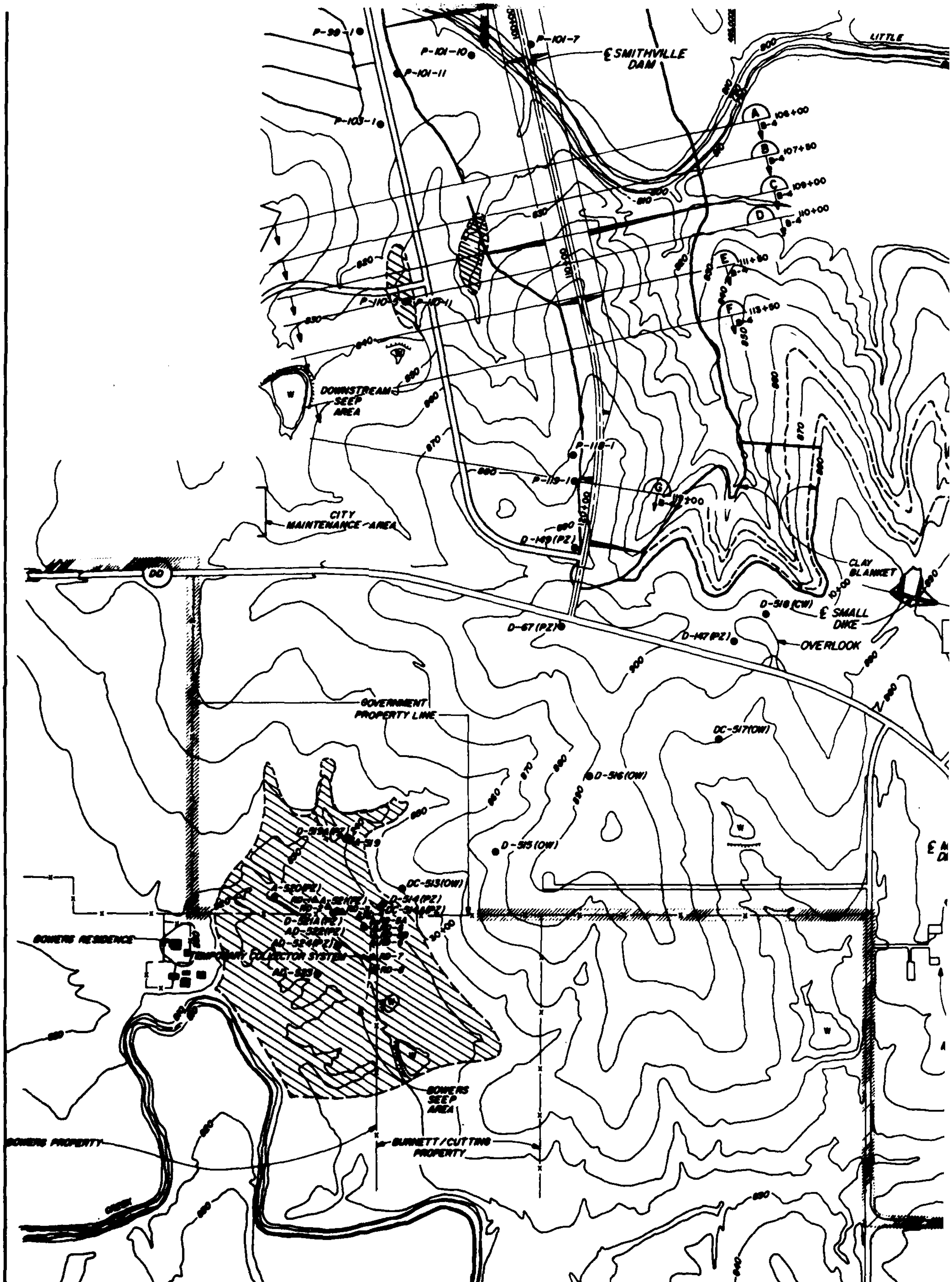
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J.G.M.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by: R.S.T.	OBSERVATION DEVICES PLAN VIEW AND SCHEDULE
Checked by: J.G.M.	Scale: AS SHOWN
Submitted by: F.C.W.	MARCH 1987
	File No.: RP-3-1763

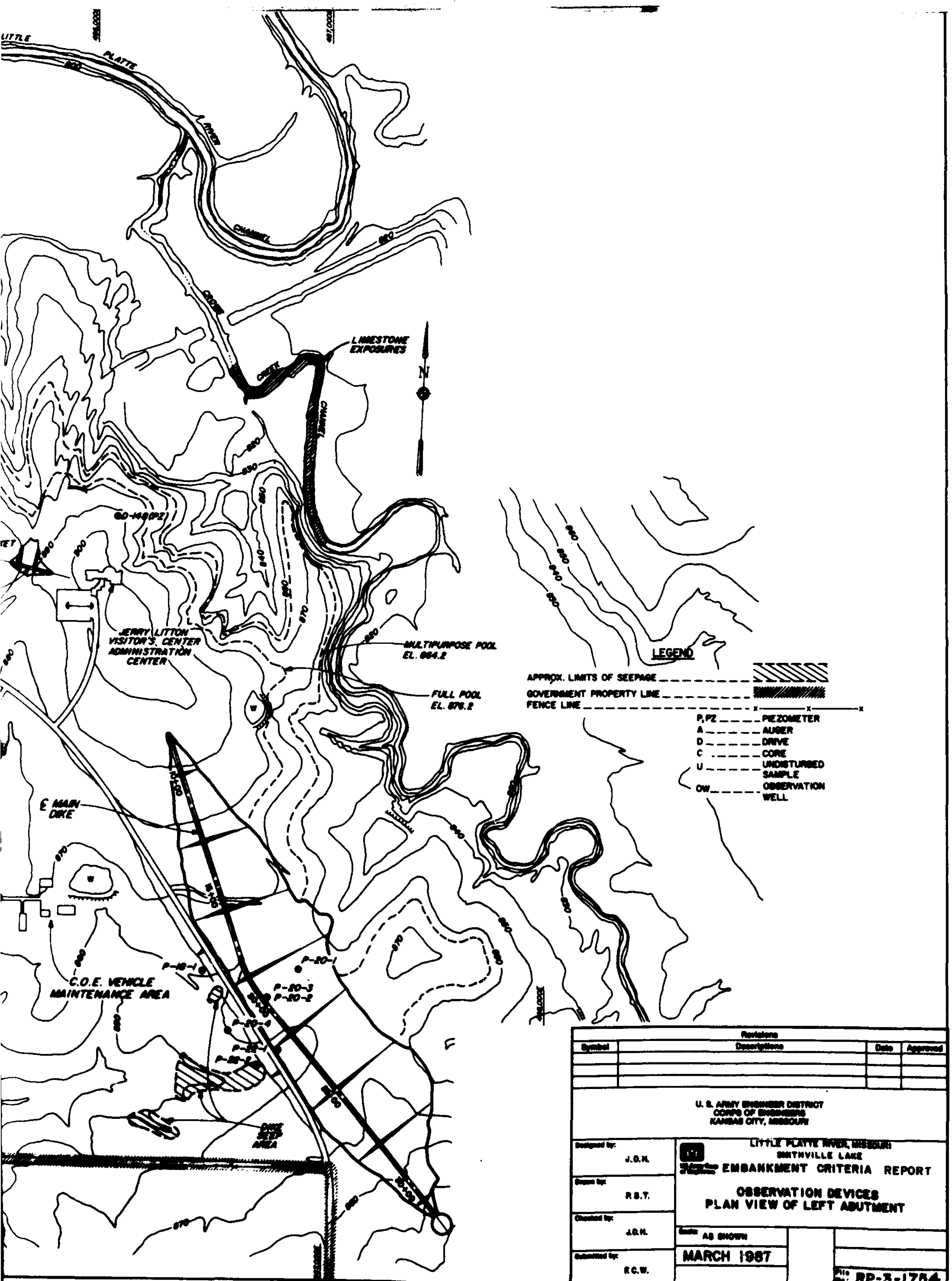
SUPPLEMENT NO 1 PLATE NO. 3

1 - -

2



P-99-1
P-101-10
P-101-11
P-101-7
℄ SMITHVILLE DAM
LITTLE
P-103-1
106+00
107+50
109+00
110+00
111+50
113+50
P-110-11
DOWNSTREAM SEEP AREA
P-118-1
CITY MAINTENANCE AREA
D-149 (PZ)
CLAY BLANKET
D-518 (OW)
℄ SMALL DIKE
D-67 (PZ)
D-147 (PZ)
OVERLOOK
GOVERNMENT PROPERTY LINE
DC-517 (OW)
D-516 (OW)
D-515 (OW)
DC-513 (OW)
D-514 (PZ)
BOWERS RESIDENCE
TEMPORARY COLLECTOR SYSTEM
BOWERS SEEP AREA
BOWERS PROPERTY
BURNETT/CUTTING PROPERTY
CREEK
SCALE IN FEET
200
0
200
400



Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

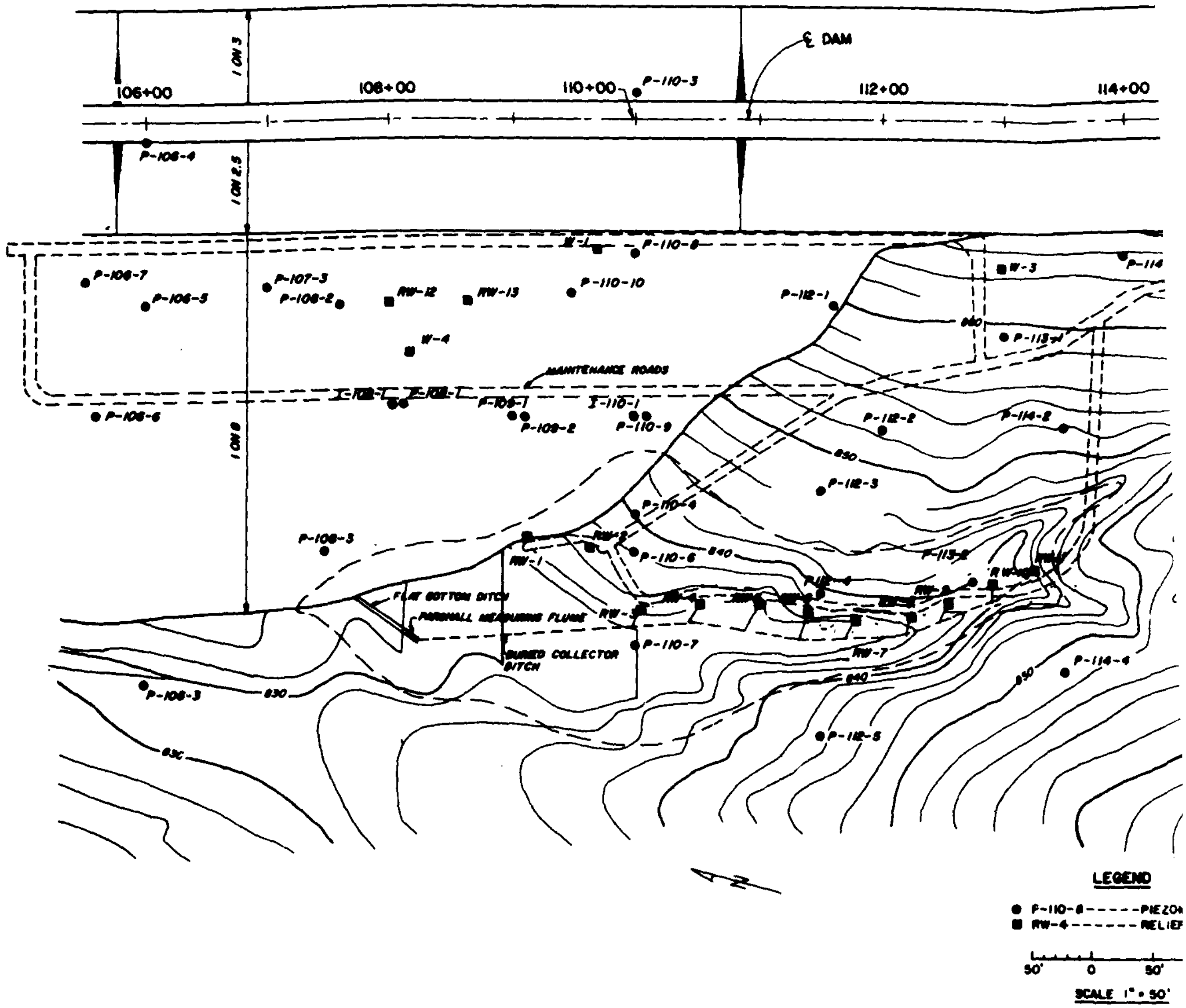
Designed by: J.D.H.
Drawn by: R.S.T.
Checked by: J.D.H.
Submitted by: R.C.W.

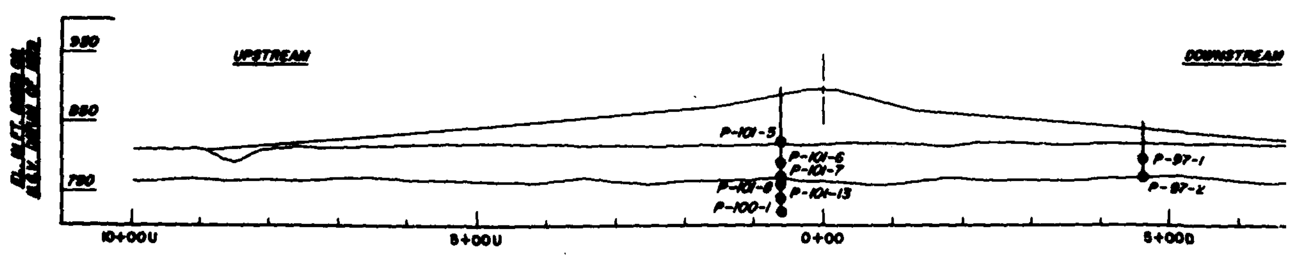
LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT

OBSERVATION DEVICES
PLAN VIEW OF LEFT ABUTMENT

Scale: AS SHOWN
MARCH 1987
File No.: RP-3-1754

SUPPLEMENT NO. 1 PLATE NO. 4



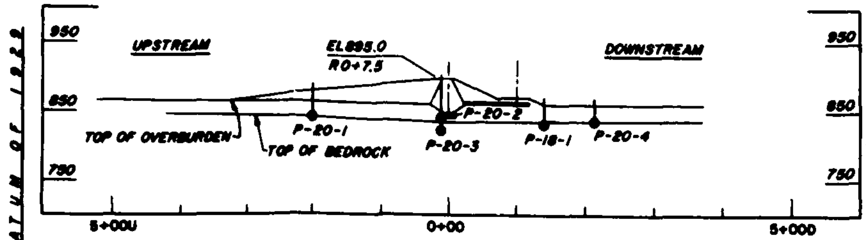


DIAGONAL ALONG LINE OF DEEPEST BEDROCK

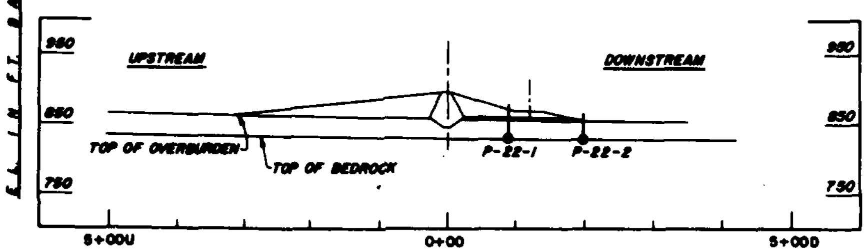
LEGEND

-----PIEZOMETERS
-----RELIEF WELLS

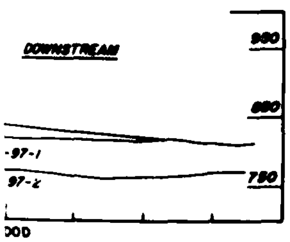
0 50' 100'
SCALE 1" = 50'



SMITHVILLE MAIN DIKE-SECTION AT STA 20+00



SMITHVILLE MAIN DIKE-SECTION AT STA 22+00



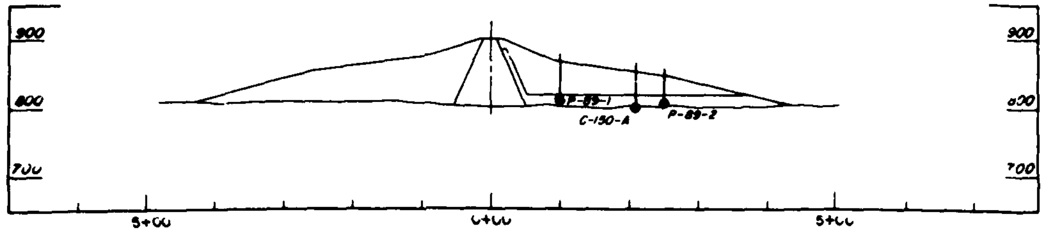
Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

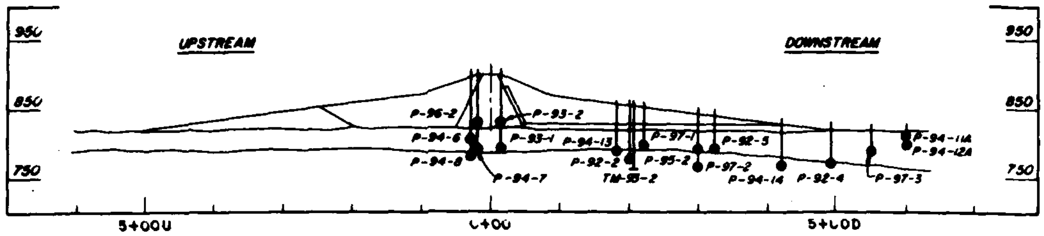
Designed by: J. D. H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT INSET FROM PRECEDING PLATE PLAN OF EXPLORATIONS & PROFILE OF PZ'S ON LEFT ABUTMENT
Drawn by: R. S. T.	
Checked by: J.D.H.	Scale: AS SHOWN
Submitted by: F. C. W.	MARCH 1987
	File No.: RP-3-1755

SUPPLEMENT NO. 1 PLATE NO. 5

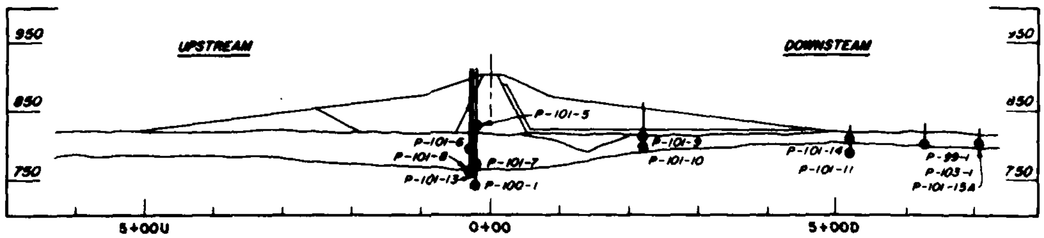
ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929



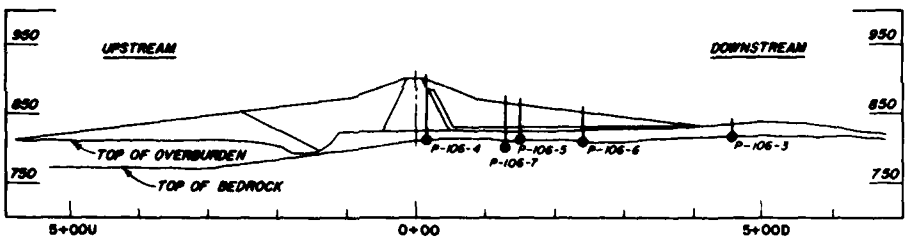
STATION 89+00



STATION 94+00

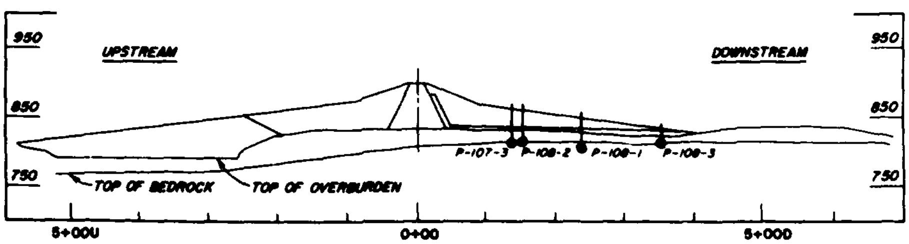


STATION 101+00



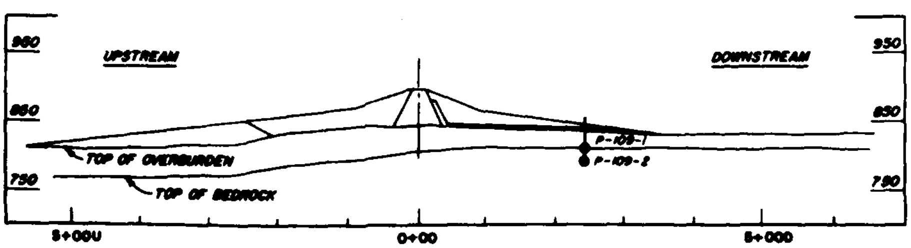
STATION 106+00

SECTION A



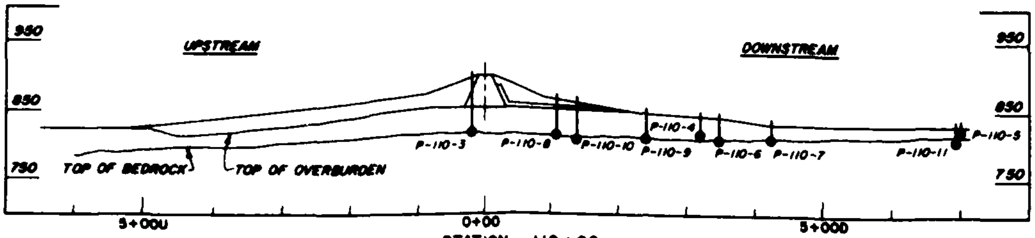
STATION 107+50

SECTION B



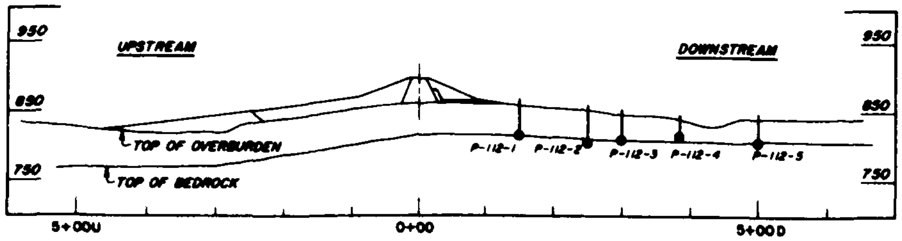
STATION 109+00

SECTION C



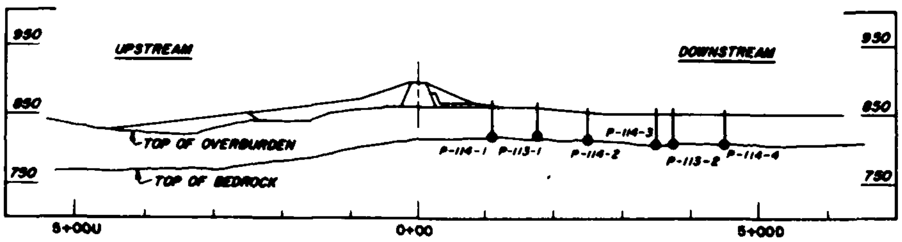
STATION 110+00

SECTION D



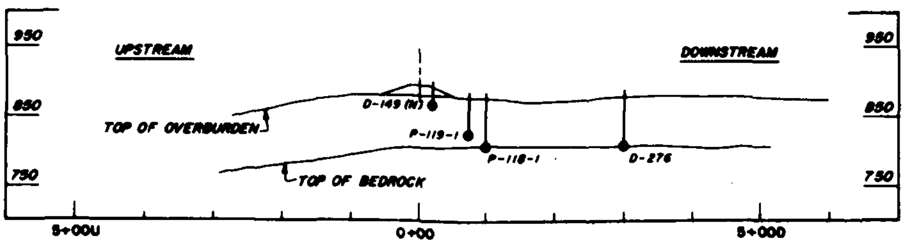
STATION 111+50

SECTION E



STATION 113+50

SECTION F



STATION 119+00

SECTION G

NOTE:
1. CROSS SECTIONS SHOWING LOCATION OF P.Z.'S AT 10 STATIONS ACROSS THE DAM. LOCATION SECTIONS A THROUGH G SHOWN ON PLATE NO. 4.

Revisions

Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT

INSTRUMENTATION CROSS SECTIONS

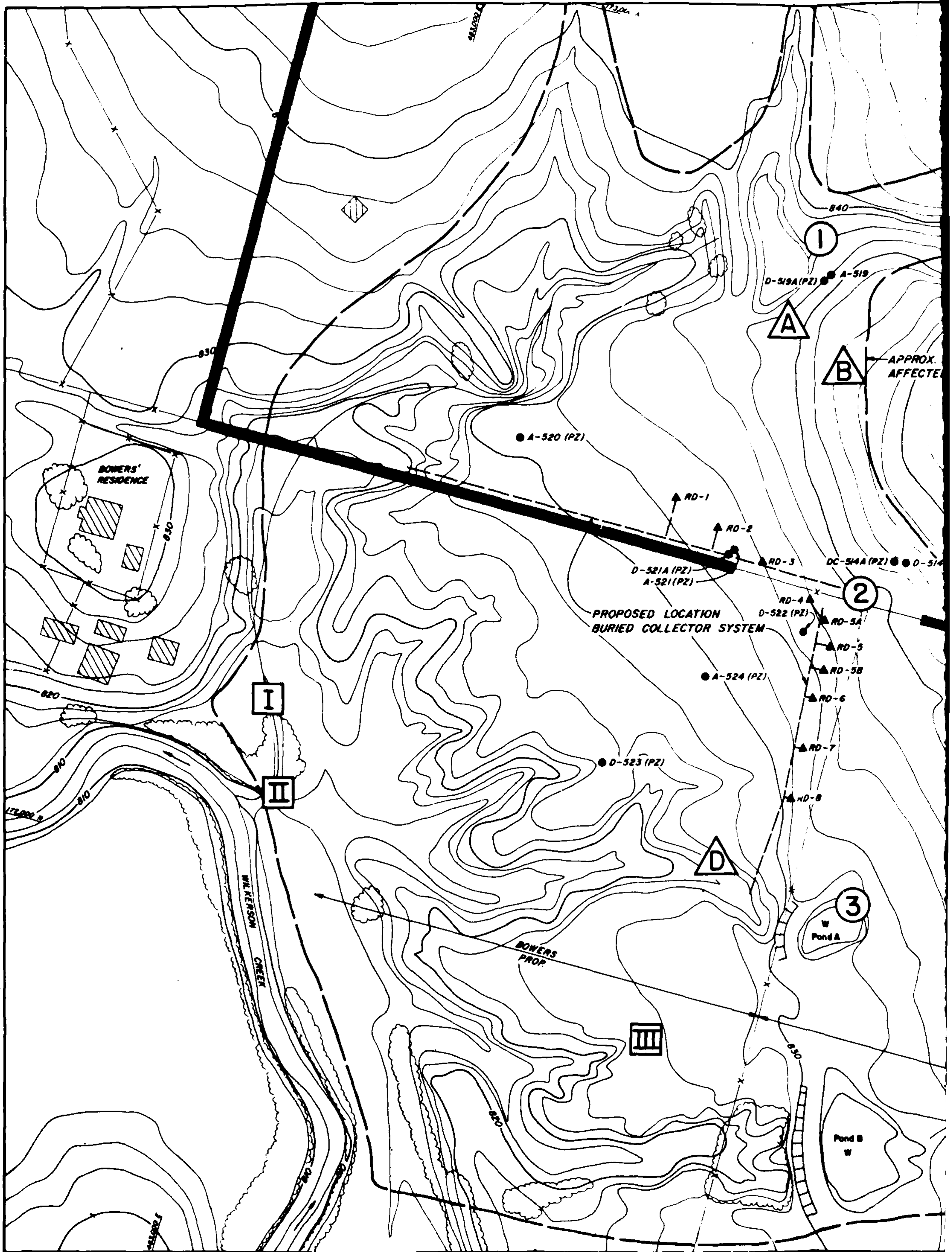
Designed by: J.B.H.
Drawn by: R.S.T.
Checked by: J.B.H.
Submitted by: F.C.M.

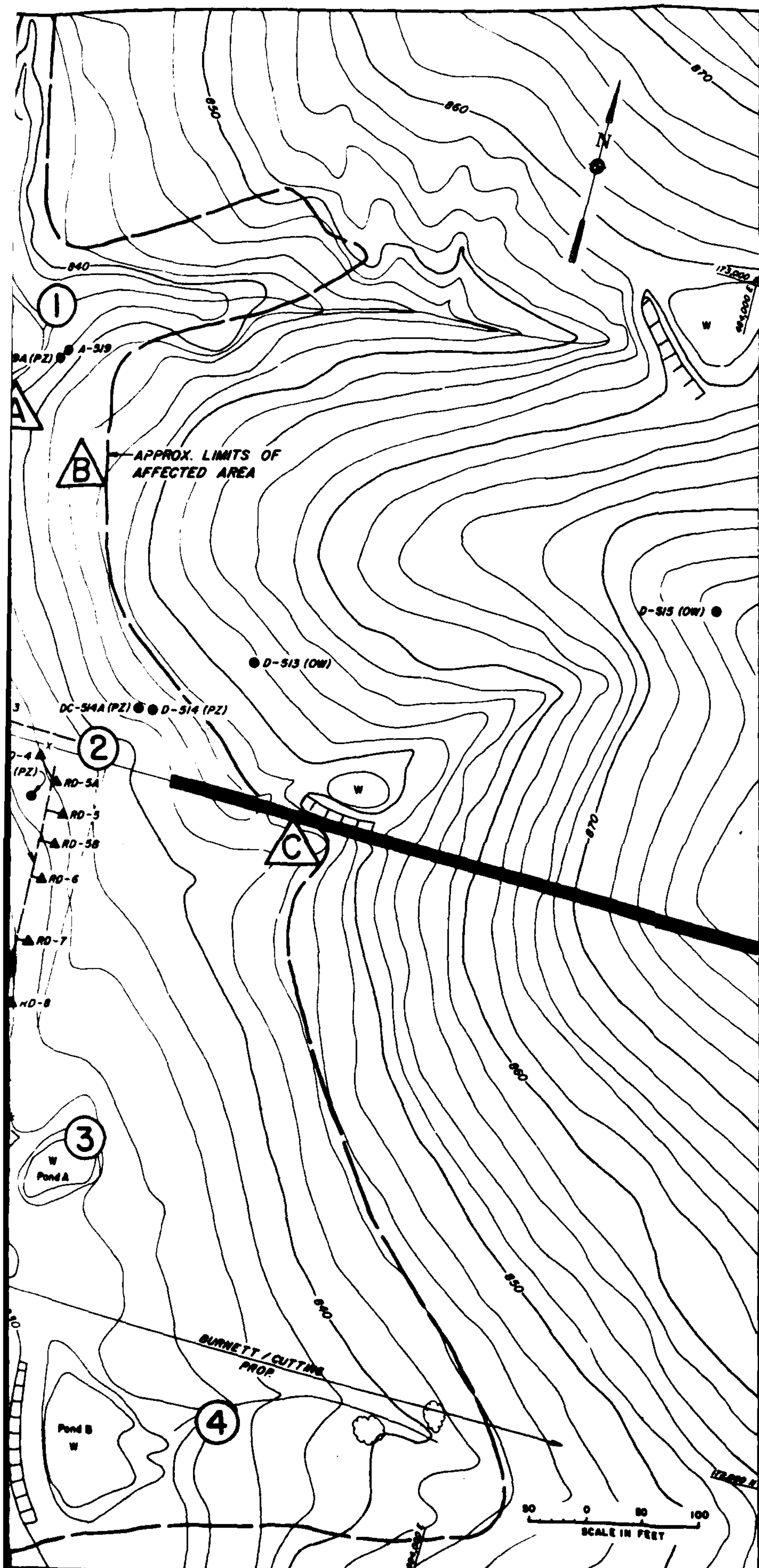
Scale: AS SHOWN

MARCH 1987

File No.: RP-3-1756

SUPPLEMENT NO. 1 PLATE NO. 6


840
830
820
810
BOWERS' RESIDENCE
A-520 (PZ)
RD-1
RD-2
RD-3
D-521A (PZ)
A-521 (PZ)
PROPOSED LOCATION
BUIRED COLLECTOR SYSTEM
D-519A (PZ)
A-519
APPROX.
AFFECTE
DC-514A (PZ)
RD-4
D-522 (PZ)
RD-5A
RD-5
RD-5B
RD-6
RD-7
RD-8
A-524 (PZ)
D-523 (PZ)
I
II
III
A
B
D
1
2
3
W
Pond A
Pond B
WILKERSON CREEK
BOWERS PROP.



LEGEND

RD	RELIEF DRAIN
(OW)	OBSERVATION WELL
(PZ)	PIEZOMETER
▬	GOVT. PROP. LINE
①	SEEPAGE - FLOWING
△	SEEPAGE - DAMP
☒	EROSION - SEVERE HEADCUTTING

Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J.M.M	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT	
Drawn by: J.W.P.	BOWERS SEEP AREA	
Checked by: J.D.H.	Scale: AS SHOWN	
Submitted by: F.C.W.	MARCH 1987	File No. RP-3-1757

SUPPLEMENT NO. I PLATE NO. 7

GEOLOGIC COLUMN - LEFT ABUTMENT, SMITHVILLE DAM

OVERBURDEN variable thickness, lean clays. Silty clays w/occasional lenses of fine sand & occasional fine to coarse gravel, underlain by clayey silts & fine to medium sands w/clay & occasional gravel. Clayey, sandy gravels w/ limestone chert & quartzite cobbles & boulders.

WYANDOTTE FM - ISLAND CREEK SHALE 26'
Soft, platy, sl clayey, calcareous, few phosphatic nodules in upper 15', lt gray to gray. Soft to v soft, clayey, w/ high angle fractures (approximately 15 to 18' above base of Island Creek). Moderately hard to hard, dense, medium to thin bedded, brown-gray limestone w/ few small open vugs & occasional wavy shale parting (approximately 5 to 7' above base of Island Creek).

WYANDOTTE FM - ARGENTINE LIMESTONE 5'
Moderately hard to hard, dense, thin wavy bedded, brown-gray, w/ numerous soft dk gray wavy shale partings & bands.

WYANDOTTE FM - QUINDARO SHALE 0.5'
Soft, very calcareous, gray, fossiliferous.

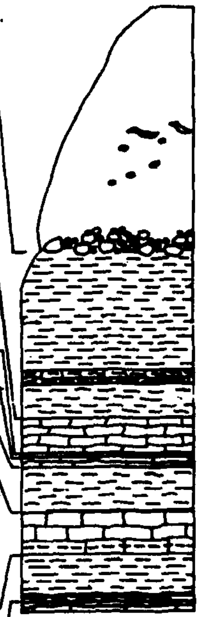
WYANDOTTE FM - FRISBIE LIMESTONE 1'
Moderately hard to hard, dense, thick-bedded, tubular & fossiliferous inclusions, gray-brown.

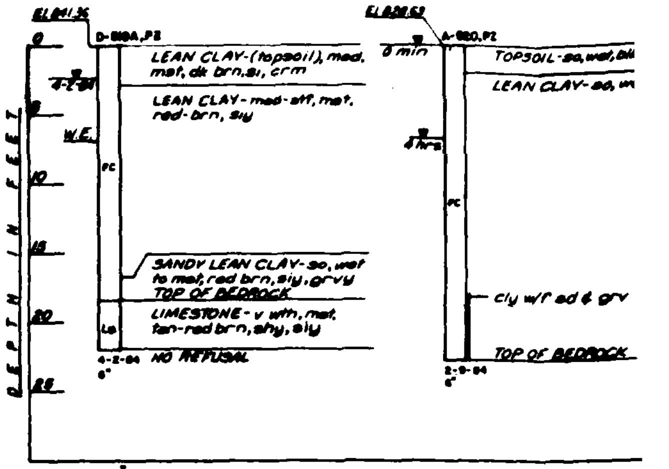
LANE FM - SHALE average thickness 7'
Soft, fissile, slightly calcareous, dark gray.

IOLA FM - RAYTOWN LIMESTONE 6'
Moderately hard to hard, crystalline to dense fossiliferous & massive in upper part. Thin-bedded w/interlaminated soft, calcareous shale w/persistent soft to v soft clayey shale (approximately 1 to 1.5' above base of Raytown) overlying moderately hard, dense to finely crystalline fossiliferous limestone w/ shaly partings in lower part.

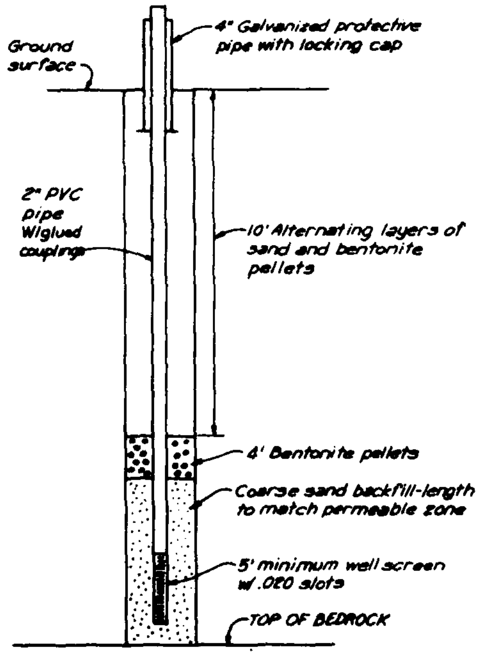
IOLA FM - MUNCIE CREEK SHALE 7'
Soft to moderately hard w/ occasional v soft thin laminae, platy, dk gray to black, calcareous, coal to black carbonaceous sh near base.

IOLA FM - PAOLA LIMESTONE 1'
Moderately hard to hard, finely crystalline, massive w/ occasional argillaceous partings, gray fossiliferous.

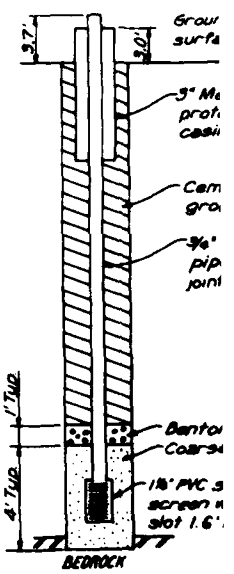




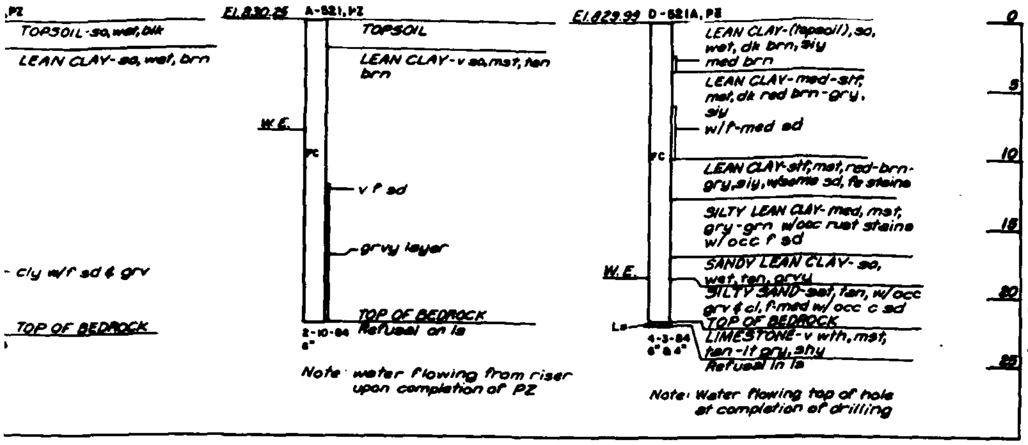
alt	alternating	dmp	
ang	angular	dol (c)	
an	anhydrite	ext	
ar	argillaceous	f (y)	
bdd	bed, bedded, bedding	fe	
bdr	bedrock	fis	
bky	blocky	fm	
bl	blue	fos (s)	
bld	boulder	frac (d)	
blk	black	frag (d)	
brec (d)	breccia, (brecciated)	fri	
brk	broken	fsl	
brn	brown	gr	
c	coarse	grn	
calc	calcareous	grn	
carb	carbonaceous	grv (y)	
cav	cavity	gry	
cbl	cobble	gyp	
cht	chert	hs	
circl	circulation	hd	
cl (y)	clay, (clayey)	hkd	
cld	closed	hor	
cmtd	cemented	intbd	
col	columnar	incl	
conc	concretions	intlam	
cong	conglomerate	irr	
crm	crumbly	jt (s)	
d	dense	lo	
dk	dark	lam (d)	



TYPICAL 2-INCH INSTALLATION OBSERVATION WELL



TYPICAL 3/4-INCH PIEZ... INSTALLATION



UNIFIED SOIL CLASSIFICATION SYSTEM

GW	Well graded gravels, gravel-sand mixtures, little or no fines.	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts w/slight plasticity.
GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
GM	Silty gravels, gravel-sand-silt mixtures.	OL	Organic silts and organic silty clays of low plasticity.
GC	Clayey gravels, gravel-sand-clay mixtures.	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
SW	Well-graded sands, gravelly sands, little or no fines.	CH	Inorganic clays of high plasticity, fat clays.
SP	Poorly graded sands or gravelly sands, little or no fines.	OH	Organic clays of medium to high plasticity, organic silts.
SM	Silty sands, sand-silt mixtures.	Pt	Peat and other highly organic soils.
SC	Clayey sands, sand-clay mixtures.		

Classification from actual laboratory tests where LL and PI are shown.
Dual classification, where used, is in accordance with the Unified Soil Classification System.
For details on the Unified Soil Classification System, See Waterways Experiment Station Technical Memorandum No. 3-357 dated March 1953 and revised in 1960.

DETACHED BORINGS

ABBREVIATIONS

dmp	damp	lea	leached	rnd (d)	round, (rounded)
dol (c)	dolomite, (dolomitic)	lig	lignite	sat	saturated
ext	extremely	ls	limestone	scat	scattered
f (y)	fine, (finely)	lt	light	sd (y)	sand, (sandy)
fe	iron	lo	loose	sev	several
fld	filled	L.C.	lost core	sh (y)	shale, (shaly)
fm	firm	L.D.W	lost drill water	si (y)	silt, (silty)
fos (s)	fossil, (fossiliferous)	med	medium	sls	siltstone
frac (d)	fractures, (fractured)	mic	micaceous	sl	slightly
frag (d)	fragments, (fragmented)	min	mineralized	silcs	siliceous
fri	friable	mod (y)	moderate, (moderately)	slks	slickensides
fsl	fissile	mot	mottled	so	soft
gr	grain	mas	massive	sol (d)	solution, (solutionized)
gra	gradation	mst	moist	ss	sandstone
grn	green	mtl	material	st (g)	stained, (staining)
grv (y)	gravel (gravelly)	mtx	matrix	stf	stiff
gry	gray	nod	nodules	sty	stylolitic
gyp	gypsum	num	numerous	v	very
ha	high angle	occ (y)	occasional, (occasionally)	vert	vertical
hd	hard	op	open	vug	vuggy
hld	healed	or	orange	w	water
hor	horizontal	org	organic	w/	with
inbd	interbedded	par	partially	wth	weathered
incl	inclusions	pit	pit, pitted, pitting	wht	white
inlam	interlaminated	pl	plastic	x-bdd	crossed-bedded
irr	irregular	ply	platy	xln	crystalline
jt (s)	joint, (joints)	pln	plane	y	yellow
la	low angle	ptg (s)	parting, (partings)		
lam (d)	laminae, (laminated)	qtz (e)	quartz, (quartzite)		

When used as log symbol first letter is capitalized

TERMS FOR CONSISTENCY OF SOIL AND HARDNESS OF BEDROCK

SOIL

Consistency	Estimated Unconfined Compressive Strength (Tons per square foot)
Very soft	< 0.25
Soft	0.25—0.5
Medium	0.5—1.0
Stiff	1.0—2.0
Very stiff	2.0—4.0
Hard	> 4.0

BEDROCK UNIT THICKNESS

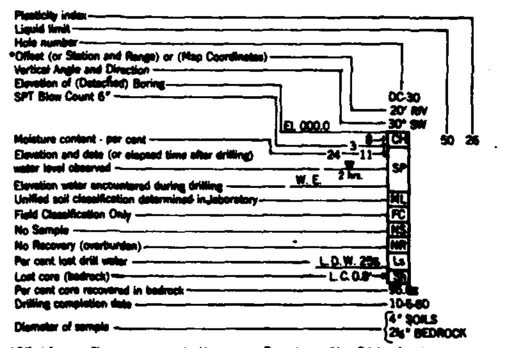
Parting	< 0.02'
Band	0.02' to 0.2'
Thin Bed	0.2' to 0.5'
Medium bed	0.5' to 1.0'
Thick bed	1.0' to 2.0'
Massive	> 2.0'

BEDROCK

SCALE OF HARDNESS

Very soft or plastic	Can be indented easily with thumb.
Soft	Can be scratched with fingernail.
Moderately hard	Can be scratched easily with knife; cannot be scratched with fingernail.
Hard	Difficult to scratch with knife.
Very Hard	Cannot be scratched with knife.

LEGEND FOR LOGS OF BORINGS



*Offset from profile or section may be Upstream or Downstream (U or D,) Landward or Riverward (LWD or RW), or Right or Left (R or L) as defined.

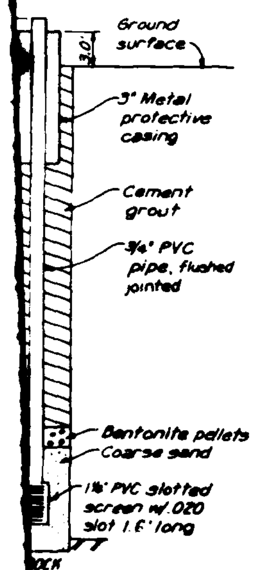
TYPE OF EXPLORATION

MAP SYMBOL

- ● Vertical boring
- 30° ● Inclined boring showing direction and vertical angle

CODE DESIGNATION

- D Drive sample hole
- C Core hole
- TP Test pit (includes power auger 24" or larger diameter)
- U Undisturbed sample hole
- A Auger hole-hand or power auger less than 24" diameter.
- NS Not Sampled (Field Classification from cuttings only)
- FS Field Section of outcrop.
- OW Observation well
- RO Relief drain
- P Piezometer no.
- PZ Piezometer



3/4-INCH PIEZOMETER INSTALLATION

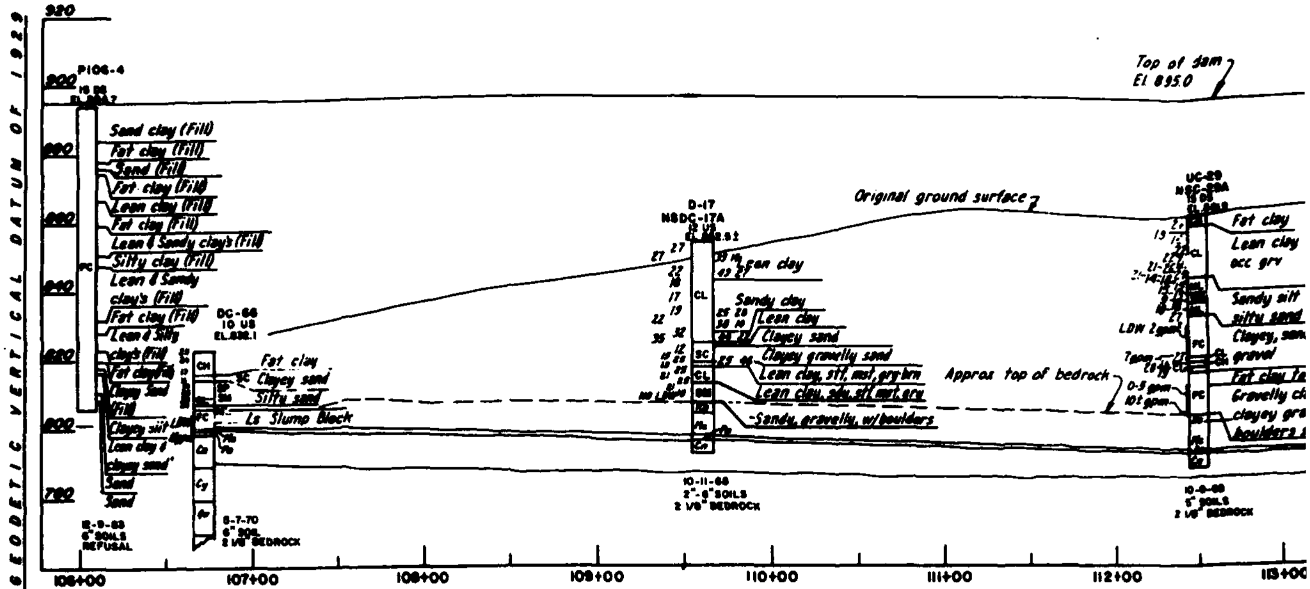
Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

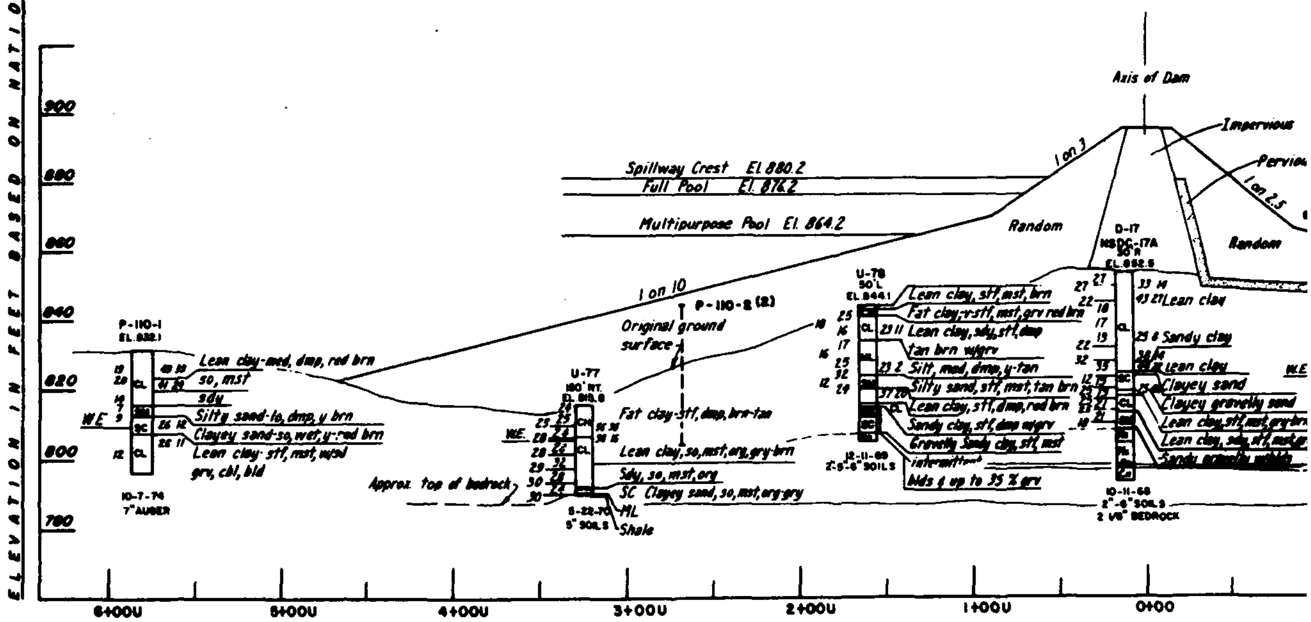
Designed by: J.M.M.	LITTLE PLATTE RIVER, MISSOURI-SMITHVILLE LAKE
Drawn by: R.L.D.	EMBANKMENT CRITERIA REPORT
Checked by: J.D.H.	GEOLOGIC COLUMN AND LEGEND, DETACHED BORINGS, AND TYPICAL INSTALLATION DIAGRAMS
Submitted by: F.C.W.	Scale: AS SHOWN
	MARCH 1987
	File No. RP-3-1758

SUPPLEMENT NO. 1 PLATE NO. 8

2

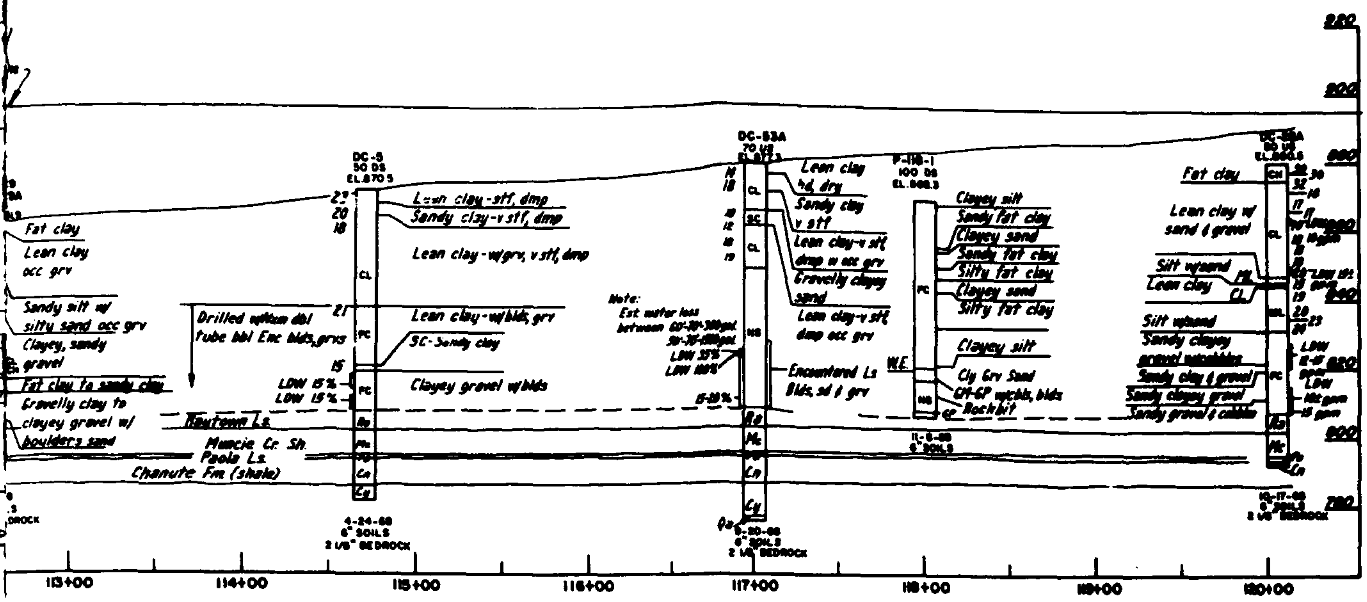


LEFT ABUTMENT DA

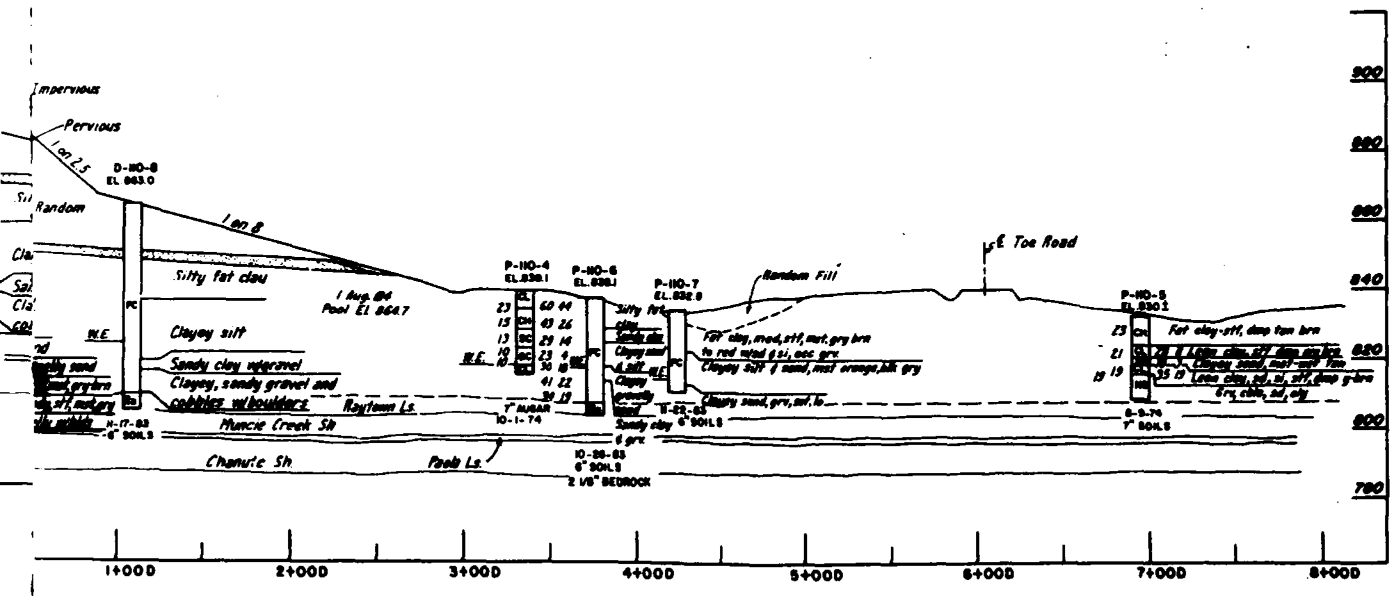


LEFT ABUTMENT
SECTION-STA. 110+00

Note:
1. Piezometric surface shown is measured in the pervious basal layer of the overburden.
2. Strip log of P-110-2 is shown on Plate No. 12.



MENT DAM AXIS PROFILE



NT

00 C 1 4

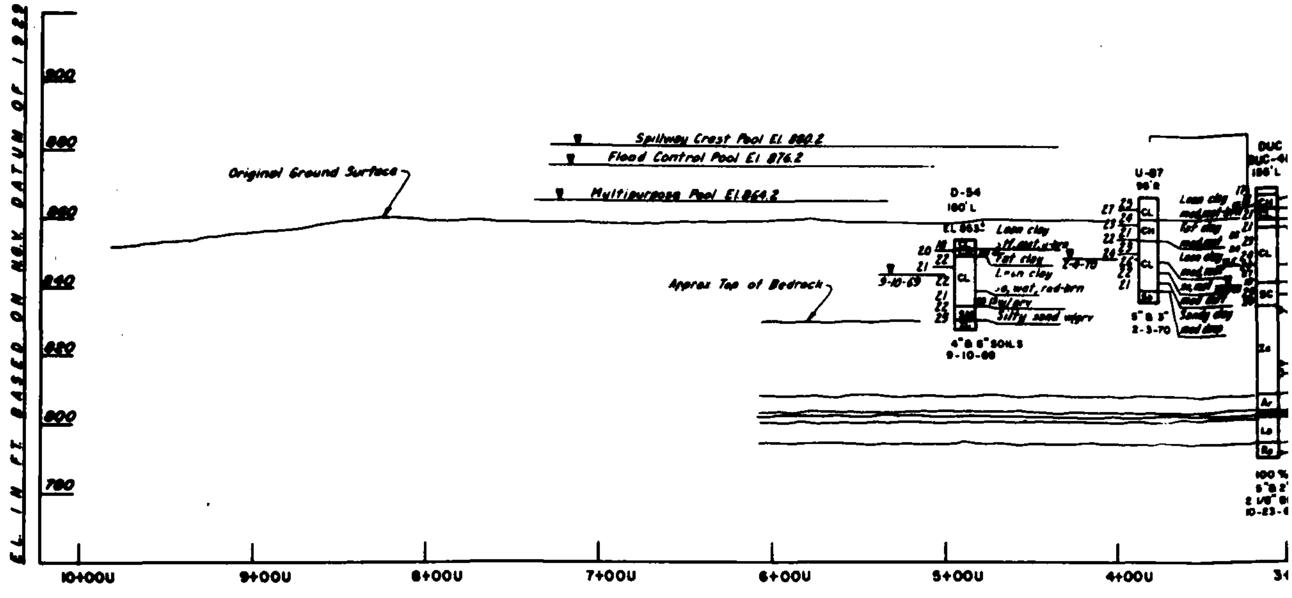
Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: W.F.L.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by: E.E.R.	LEFT ABUTMENT DAM AXIS PROFILE; EMBANKMENT SECTION-STATION 110+00
Checked by: J.B.H.	Scale: AS SHOWN
Submitted by: R.C.W.	MARCH 1987
	File No. RP-3-1759

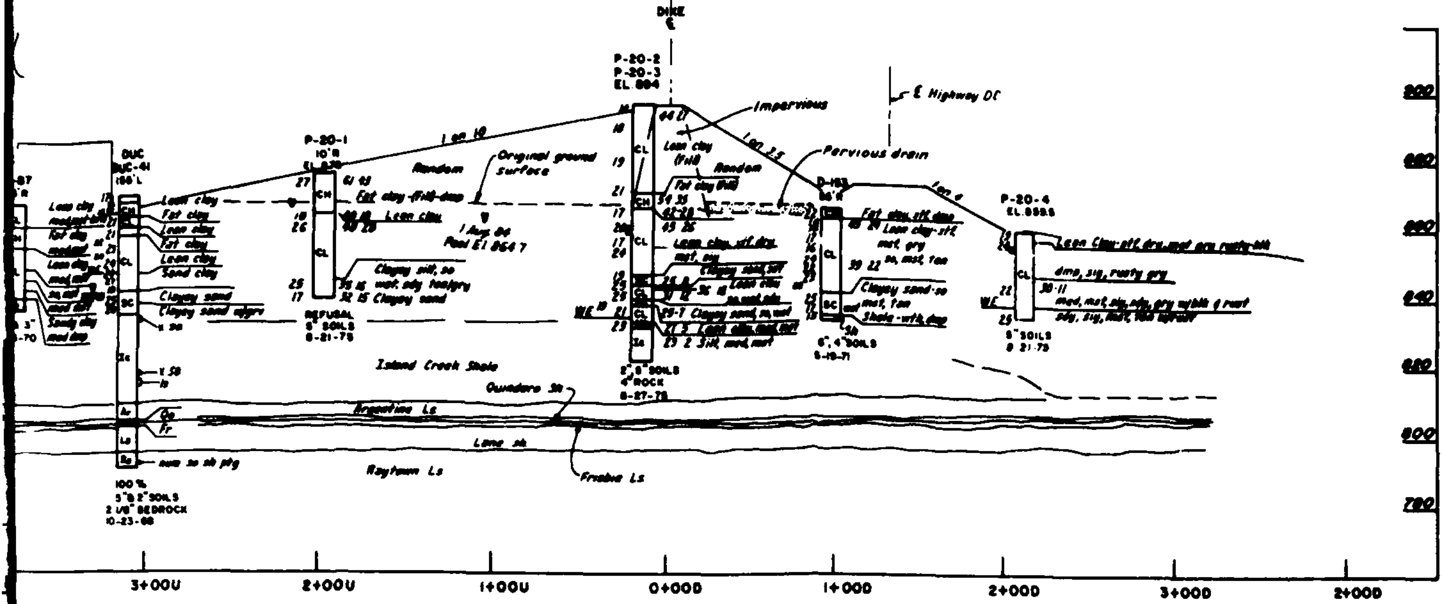
SUPPLEMENT NO. 1 PLATE NO. 9

2



MAIN DIKE

Note:
1. Piezometric surface show is measured in the pervious basal layer of the overburden



MAIN DIKE SECTION D/15

Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: W.F.L.
Drawn by: E.E.R.
Checked by: J.D.H.
Submitted by: F.C.W.

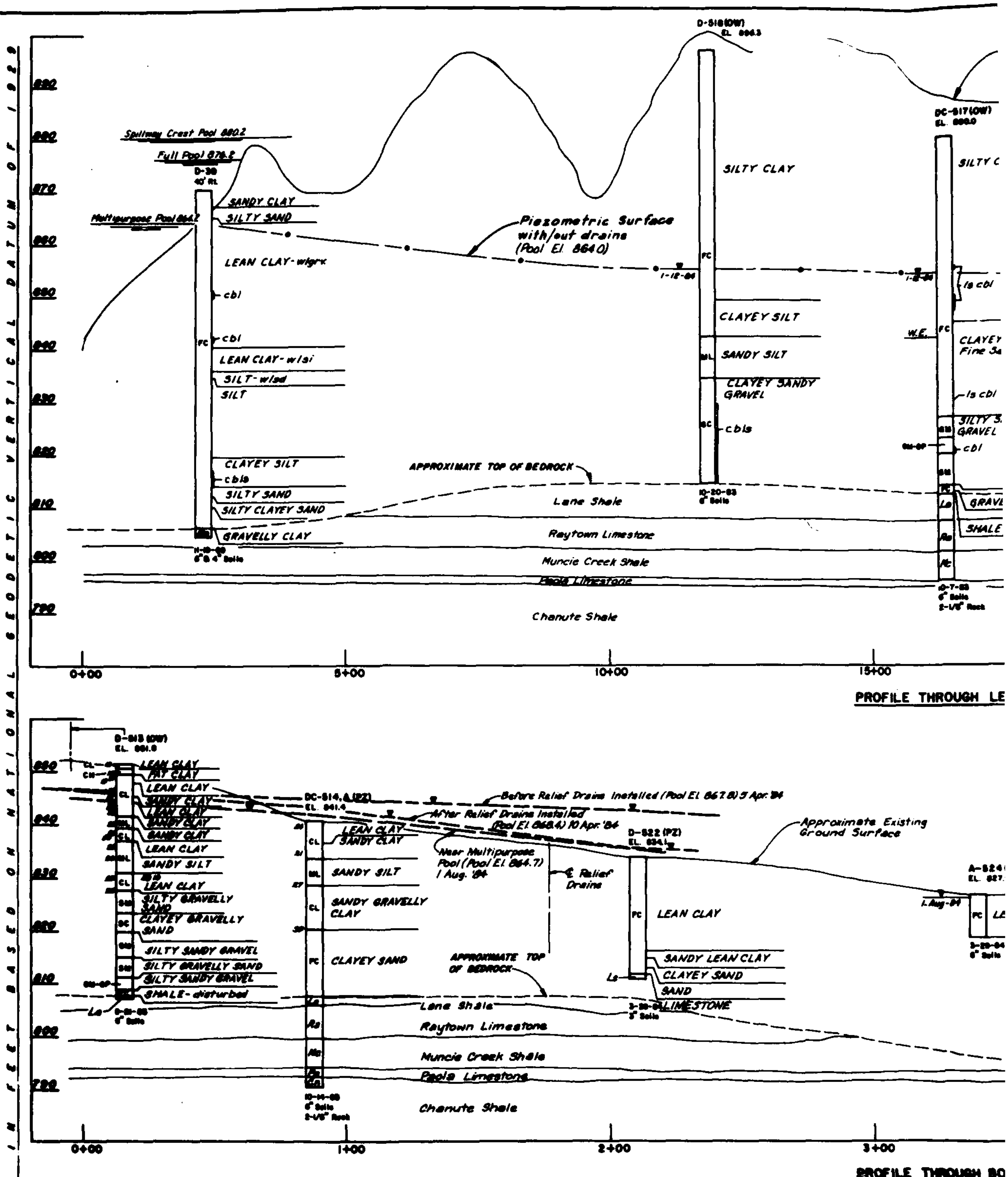
LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT

MAIN DIKE SECTION-STATION 20+00

Scale: AS SHOWN

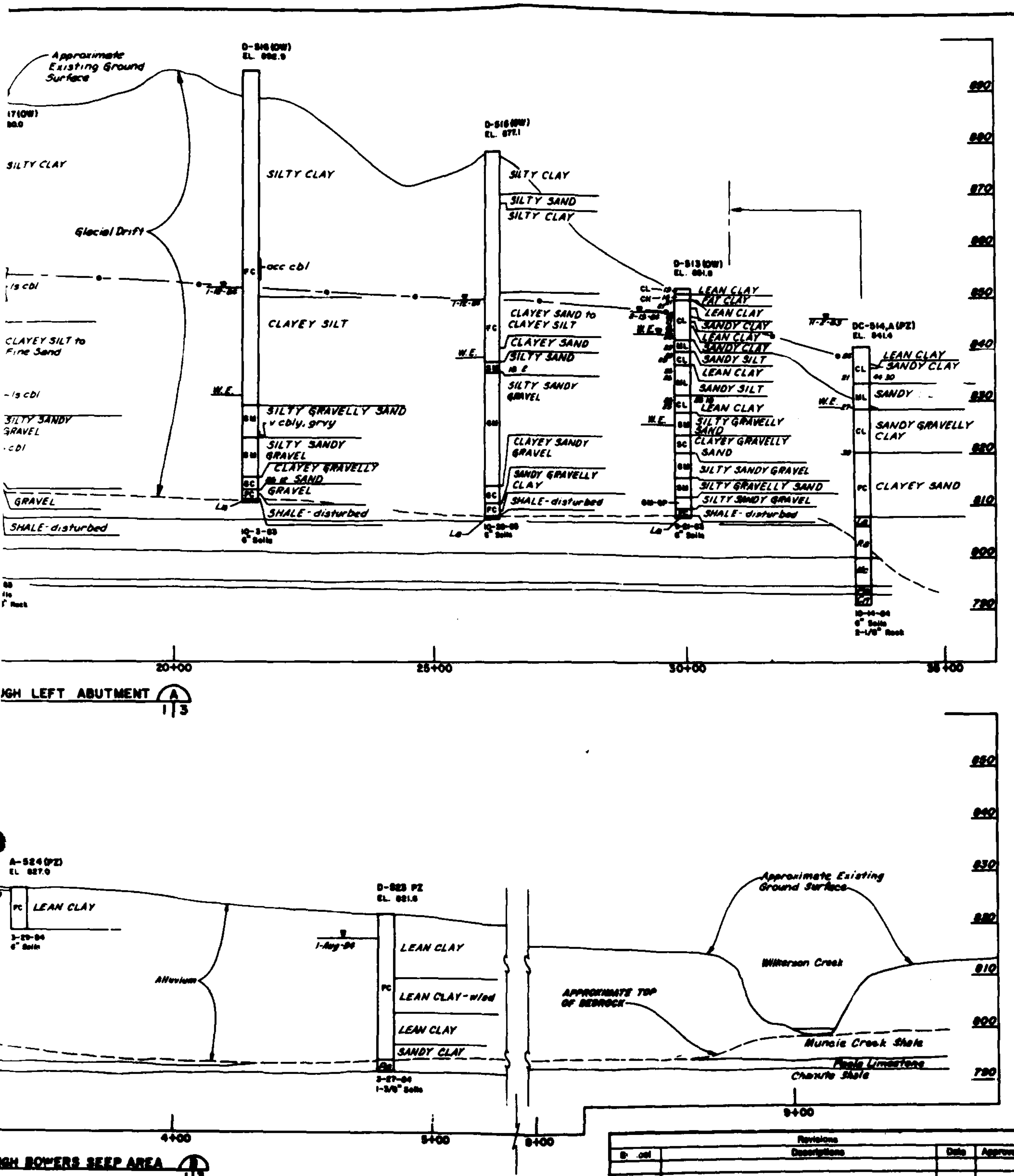
MARCH 1987

File No. RP 3 1760



Notes:

1. *Boundary between glacial drift and alluvium is not defined.*
2. *Piezometric surface shown is measured in the pervious basal layer of the overburden.*

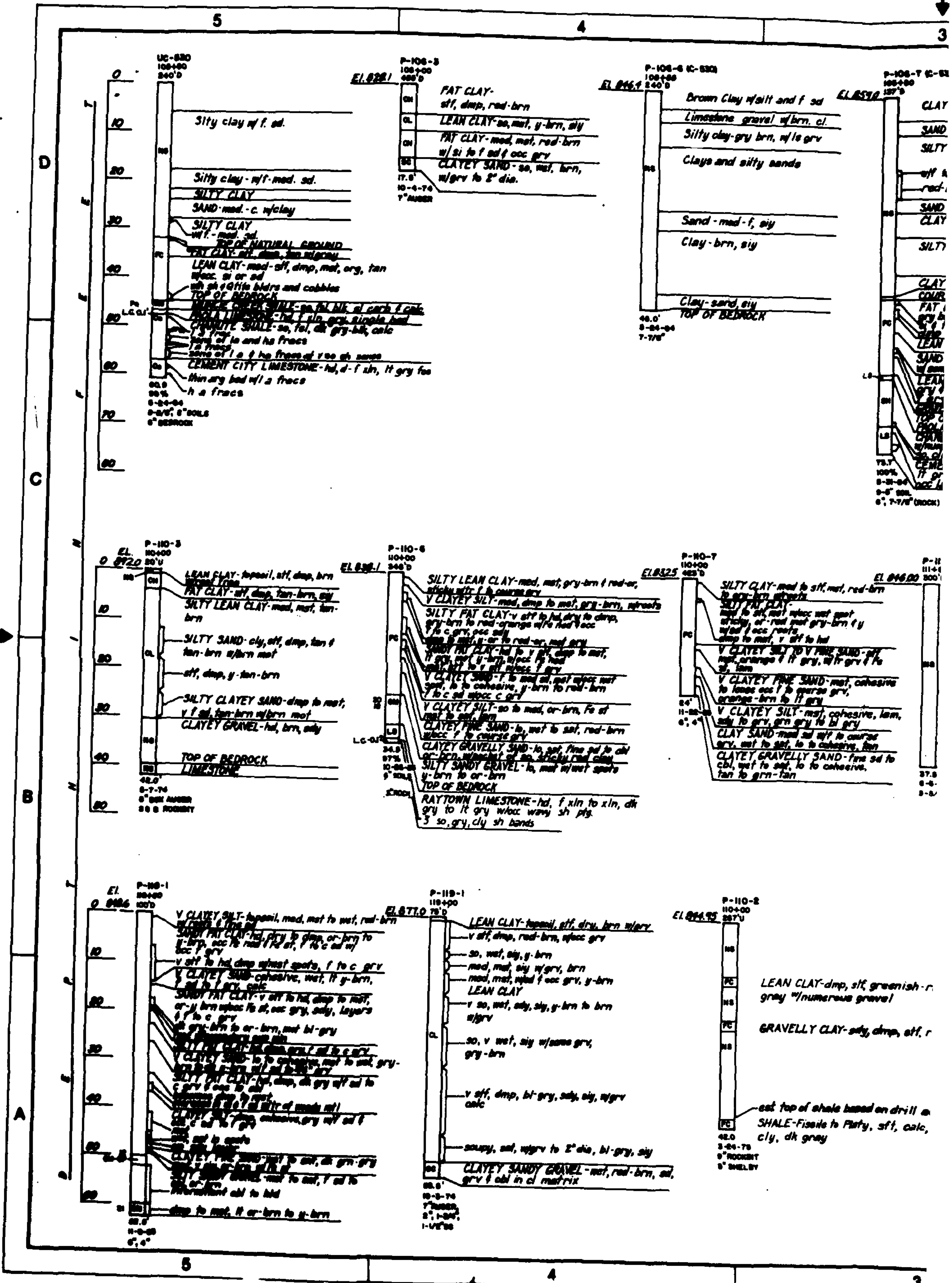


Sym	Descriptions	Date	Approved

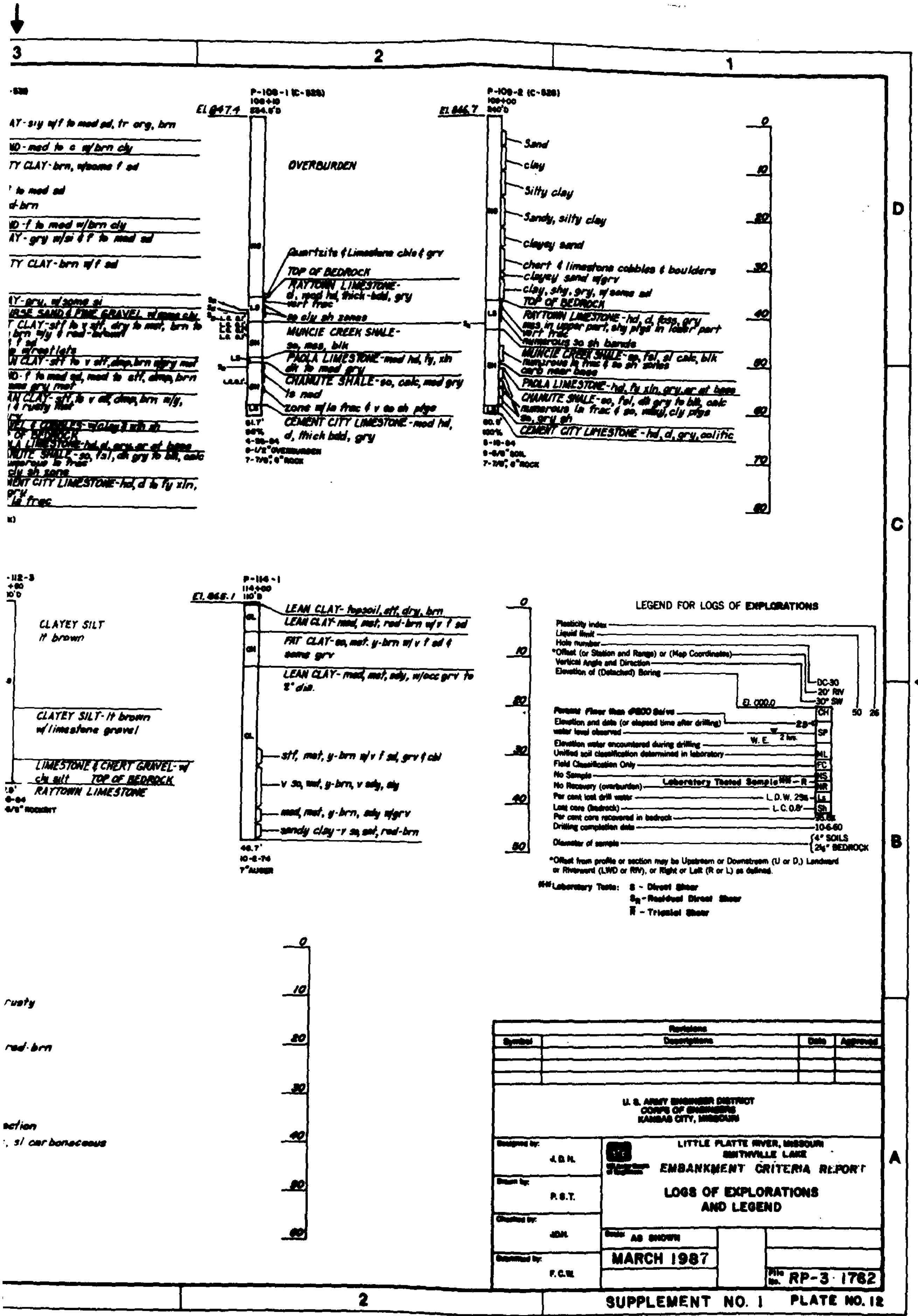
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J.M.M.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT	
Drawn by: J.S.S.	PROFILES-LEFT ABUTMENT BOWERS SEEP AREA	
Checked by: J.D.H.	Scale: AS SHOWN	
Submitted by: F.C.M.	MARCH 1987	File No. RP-3-1761

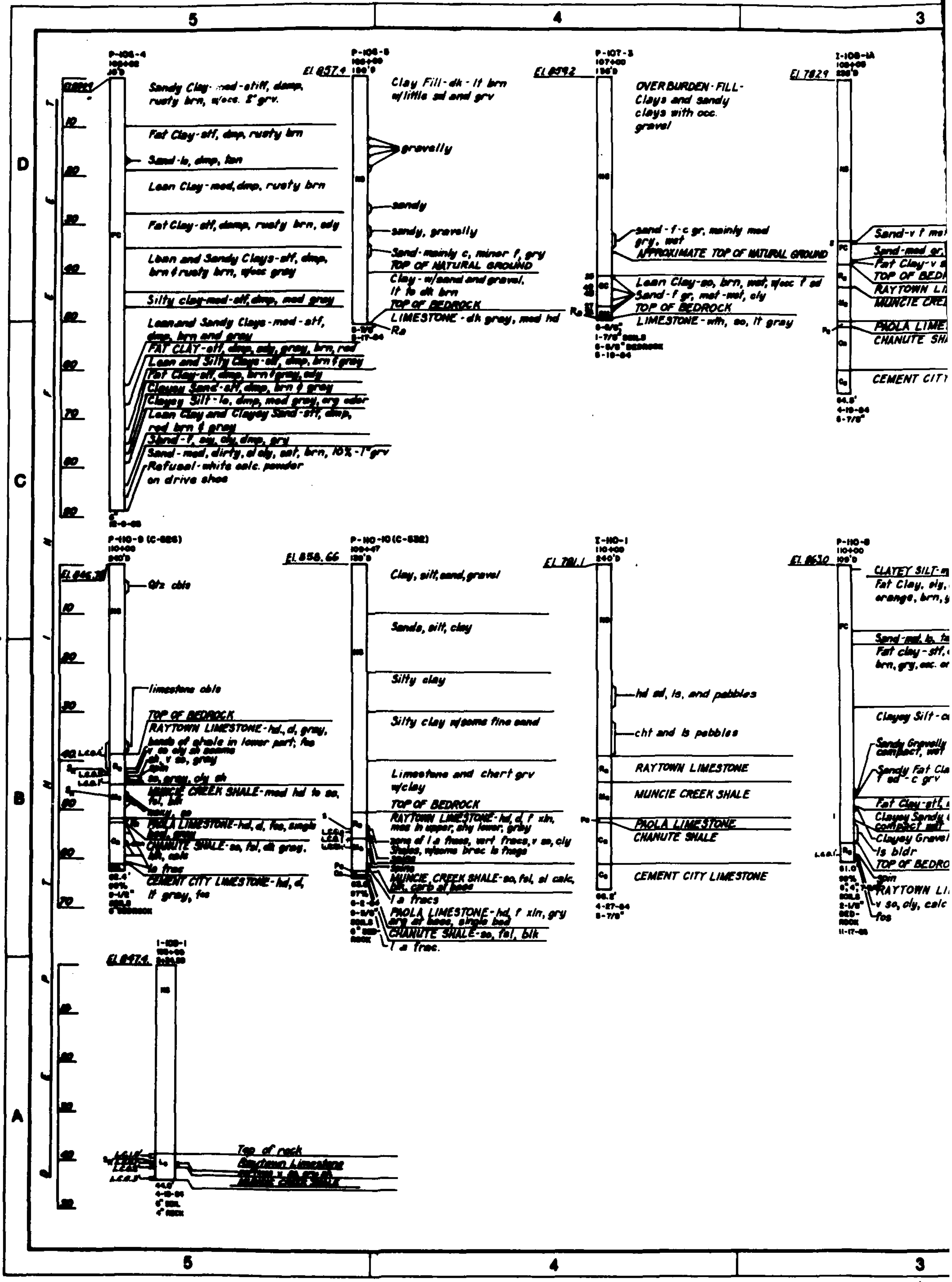
2



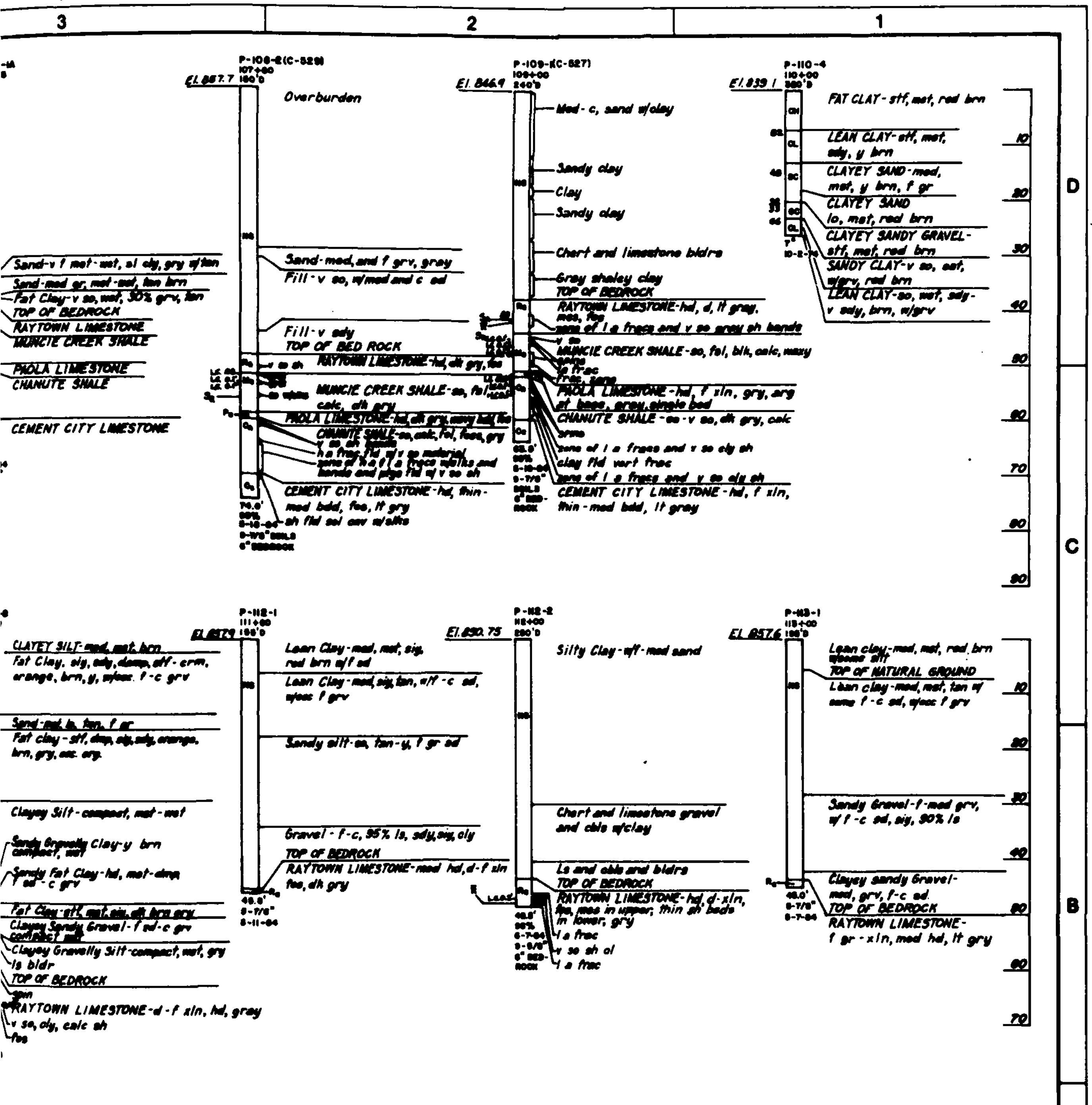
5
4
3
D
C
B
A
UC-530
P-106-3
EL 828.1
FAT CLAY-stf, dmp, red-brn
LEAN CLAY-so, mst, y-brn, sly
FAT CLAY-med, mst, red-brn w/si to f sd & occ grv
CLAYEY SAND-so, mst, brn, w/grv to 2" dia.
P-106-6 (C-530)
EL 846.4
Brown Clay w/silt and f sd
Limestone gravel w/brn. cl.
Silty clay-gry brn, w/lo grv
Clays and silty sands
Sand-med-f, sly
Clay-brn, sly
Clay-sand, sly
TOP OF BEDROCK
P-106-7
EL 859.0
Silty clay w/ f. sd.
Silty clay-w/f-med. sd.
SILTY CLAY
SAND-med.-c. w/clay
SILTY CLAY
TOP OF NATURAL GROUND
LEAN CLAY-med-stf, dmp, mst, org, tan
TOP OF BEDROCK
CEMENT CITY LIMESTONE-hd, d-f xln, lt gry fos
thin arg bed w/l a fracs
h a fracs
P-110-3
EL. 892.0
LEAN CLAY-topsoil, stf, dmp, brn
FAT CLAY-stf, dmp, tan-brn, sly
SILTY LEAN CLAY-med, mst, tan-brn
SILTY SAND-cly, stf, dmp, tan & tan-brn w/brn mot
stf, dmp, y-tan-brn
SILTY CLAYEY SAND-dmp to mst, v f sd, tan-brn w/brn mot
CLAYEY GRAVEL-hd, brn, sdy
TOP OF BEDROCK
LIMESTONE
P-110-6
EL 838.1
SILTY LEAN CLAY-med, mst, gry-brn & red-or, sticky
V CLAYEY SILT-med, dmp to mst, gry-brn
SILTY FAT CLAY-v stf to hd, dry to dmp
TOP OF BEDROCK
RAYTOWN LIMESTONE-hd, f xln to xln, dk gry to lt gry w/occ wavy sh ptg.
3 so, gry, cly sh bands
P-110-7
EL 832.5
SILTY CLAY-med to stf, mst, red-brn
V CLAYEY SILT-mst, cohesive, lam,
CLAYEY GRAVELLY SAND-fine sd to cbl, wet to sat, lo to cohesive, tan to grn-tan
EL 846.00
P-119-1
EL 877.0
LEAN CLAY-topsoil, stf, dry, brn w/grv
v stf, dmp, red-brn, w/occ grv
so, wet, sly, y-brn
med, mst, sly w/grv, brn
LEAN CLAY
so, v wet, sly w/some grv, gry-brn
v stf, dmp, bl-gry, sdy, sly, w/grv calc
soupy, sat, w/grv to 2" dia, bl-gry, sly
CLAYEY SANDY GRAVEL-mst, red-brn, sd, grv & cbl in cl matrix
P-119-1
EL 848.6
V CLAYEY SILT-topsoil, med, mst to wet, red-brn
v stf to hd, dmp w/mst spots, f to c grv
dmp to mst, lt or-brn to y-brn
P-110-2
EL 844.95
LEAN CLAY-dmp, stf, greenish-r. gray w/numerous gravel
GRAVELLY CLAY-sdy, dmp, stf, r
est top of shale based on drill
SHALE-Fissile to Platy, stf, calc, cly, dk gray



P-108-1 (C-538)
EL 847.4
OVERBURDEN
Quartzite & Limestone cbls & grv
TOP OF BEDROCK
RAYTOWN LIMESTONE-d, mod hd, thick-bdd, gry
vert frac
so cly sh zones
MUNCIE CREEK SHALE-so, mas, blk
PAOLA LIMESTONE-mod hd, fy, xln
dk to med gry
CHANUTE SHALE-so, calc, med gry to red
zone w/lo frac & v so sh plgs
CEMENT CITY LIMESTONE-mod hd, d, thick bdd, gry
P-108-2 (C-538)
EL 846.7
Sand
clay
Silty clay
Sandy, silty clay
clayey sand
chert & limestone cobbles & boulders
clayey sand w/grv
clay, shy, gry, w/some si
TOP OF BEDROCK
RAYTOWN LIMESTONE-hd, d, foss, gry
mas in upper part, shy plgs in lower part
vert frac
numerous so sh bands
MUNCIE CREEK SHALE-so, fsl, sl calc, blk
numerous lo frac & so sh zones
carb near base
PAOLA LIMESTONE-hd, fy xln, gry, ar at base
CHANUTE SHALE-so, fsl, dk gry to blk, calc
numerous lo frac & so, silty, cly plgs
so, gry sh
CEMENT CITY LIMESTONE-hd, d, gry, oolitic
P-116-1
El. 868.1
LEAN CLAY-topsoil, stf, dry, brn
LEAN CLAY-med, mst, red-brn w/v f sd
FAT CLAY-so, mst, y-brn w/v f sd & some grv
LEAN CLAY-med, mst, sdy, w/occ grv to 2" dia.
stf, mst, y-brn w/v f sd, grv & cbl
v so, wet, y-brn, v sdy, sly
med, mst, y-brn, sdy w/grv
sandy clay-v so, sat, red-brn
CLAYEY SILT lt brown
CLAYEY SILT-lt brown w/limestone gravel
LIMESTONE & CHERT GRAVEL-w/ cly silt
TOP OF BEDROCK
RAYTOWN LIMESTONE
LEGEND FOR LOGS OF EXPLORATIONS
Plasticity index
Liquid limit
Hole number
*Offset (or Station and Range) or (Map Coordinates)
Vertical Angle and Direction
Elevation of (Detached) Boring
Percent Finer than #200 Sieve
Elevation and date (or elapsed time after drilling) water level observed
Elevation water encountered during drilling
Unified soil classification determined in laboratory
Field Classification Only
No Sample
No Recovery (overburden)
Laboratory Tested Sample
Per cent lost drill water
Lost core (bedrock)
Per cent core recovered in bedrock
Drilling completion date
Diameter of sample
DC-30
20' RV
30' SW
El. 000.0
W.E. 2 hrs.
L.D.W. 29%
L.C. 0.8'
10-6-60
4" SOILS
2⅛" BEDROCK
*Offset from profile or section may be Upstream or Downstream (U or D,) Landward or Riverward (LWD or RW), or Right or Left (R or L) as defined.
Laboratory Tests: S - Direct Shear
S_R - Residual Direct Shear
R - Triaxial Shear
Revisions
Symbol
Descriptions
Date
Approved
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI
LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT
LOGS OF EXPLORATIONS AND LEGEND
Designed by: J.D.N.
Drawn by: P.S.T.
Checked by: JDN.
Submitted by: F.C.W.
Scale: AS SHOWN
MARCH 1987
File No. RP-3 1782
SUPPLEMENT NO. 1
PLATE NO. 12



5
4
3
D
C
B
A
P-106-4
EL 857.4
P-106-5
EL 859.2
P-107-3
EL 782.9
I-108-1A
Sandy Clay-med-stiff, damp, rusty brn, w/occ. 2" grv.
Fat Clay-stf, dmp, rusty brn
Sand-lo, dmp, tan
Lean Clay-med, dmp, rusty brn
Fat Clay-stf, damp, rusty brn, sdy
Lean and Sandy Clays-stf, dmp, brn & rusty brn, w/occ gray
Silty clay-med-stf, dmp, med gray
Lean and Sandy Clays-med-stf, dmp, brn and gray
FAT CLAY-stf, dmp, sdy, gray, brn, red
Lean and Silty Clays-stf, dmp, brn & gray
Fat Clay-stf, dmp, brn & gray, sdy
Clayey Sand-stf, dmp, brn & gray
Clayey Silt-lo, dmp, med gray, org odor
Lean Clay and Clayey Sand-stf, dmp, red brn & gray
Sand-f, sly, cly, dmp, gry
Sand-med, dirty, sl cly, sat, brn, 10% -1" grv
Refusal-white calc. powder on drive shoe
Clay Fill-dk-lt brn w/little sd and grv
gravelly
sandy
sandy, gravelly
Sand-mainly c, minor f, gry
TOP OF NATURAL GROUND
Clay-w/sand and gravel, lt to dk brn
TOP OF BEDROCK
LIMESTONE-dk gray, med hd
Ra
OVERBURDEN FILL-Clays and sandy clays with occ. gravel
sand-f-c gr, mainly med gry, wet
APPROXIMATE TOP OF NATURAL GROUND
Lean Clay-so, brn, wet, w/occ f sd
Sand-f gr, med-wet, cly
TOP OF BEDROCK
LIMESTONE-wht, so, lt gray
Sand-v f
Sand-med gr
Fat Clay-v
TOP OF BEDR
RAYTOWN LI
MUNCIE CRE
PAOLA LIME
CHANUTE SH
CEMENT CITY
P-110-9 (C-686)
EL 846.78
P-110-10 (C-532)
EL 858.66
I-110-1
EL 791.1
P-110-8
EL 863.0
Qtz cble
limestone cble
TOP OF BEDROCK
RAYTOWN LIMESTONE-hd, d, gray, bands of shale in lower part, fos
MUNCIE CREEK SHALE-med hd to so, fol, blk
PAOLA LIMESTONE-hd, d, fos, single bed
CHANUTE SHALE-so, fol, dk gray, blk, calc
ls frag
CEMENT CITY LIMESTONE-hd, d, lt gray, fos
Clay, silt, sand, gravel
Sands, silt, clay
Silty clay
Silty clay w/some fine sand
Limestone and chert grv w/clay
TOP OF BEDROCK
RAYTOWN LIMESTONE-hd, d, f xln, mas in upper, shy lower, grby
MUNCIE CREEK SHALE-so, fol, sl calc, blk, carb at base
l a fracs
PAOLA LIMESTONE-hd, f xln, gry, arg at base, single bed
CHANUTE SHALE-so, fol, blk
l a frac.
hd sd, ls, and pebbles
cht and ls pebbles
RAYTOWN LIMESTONE
MUNCIE CREEK SHALE
PAOLA LIMESTONE
CHANUTE SHALE
CEMENT CITY LIMESTONE
CLAYEY SILT
Fat Clay, sly, orange, brn
Sand-med, lo
Fat clay-stf, brn, gry
Clayey Silt
Sandy Gravelly compact, wet
Sandy Fat Cla f sd-c grv
Fat Clay-stf
Clayey Sandy compact
Clayey Gravel
ls bldr
TOP OF BEDRO
RAYTOWN LI
v so, cly, calc
fos
I-108-1
EL 847.4
Top of rock
Raytown Limestone
Muncie Creek Shale
10
20
30
40
50
60
70
80
90



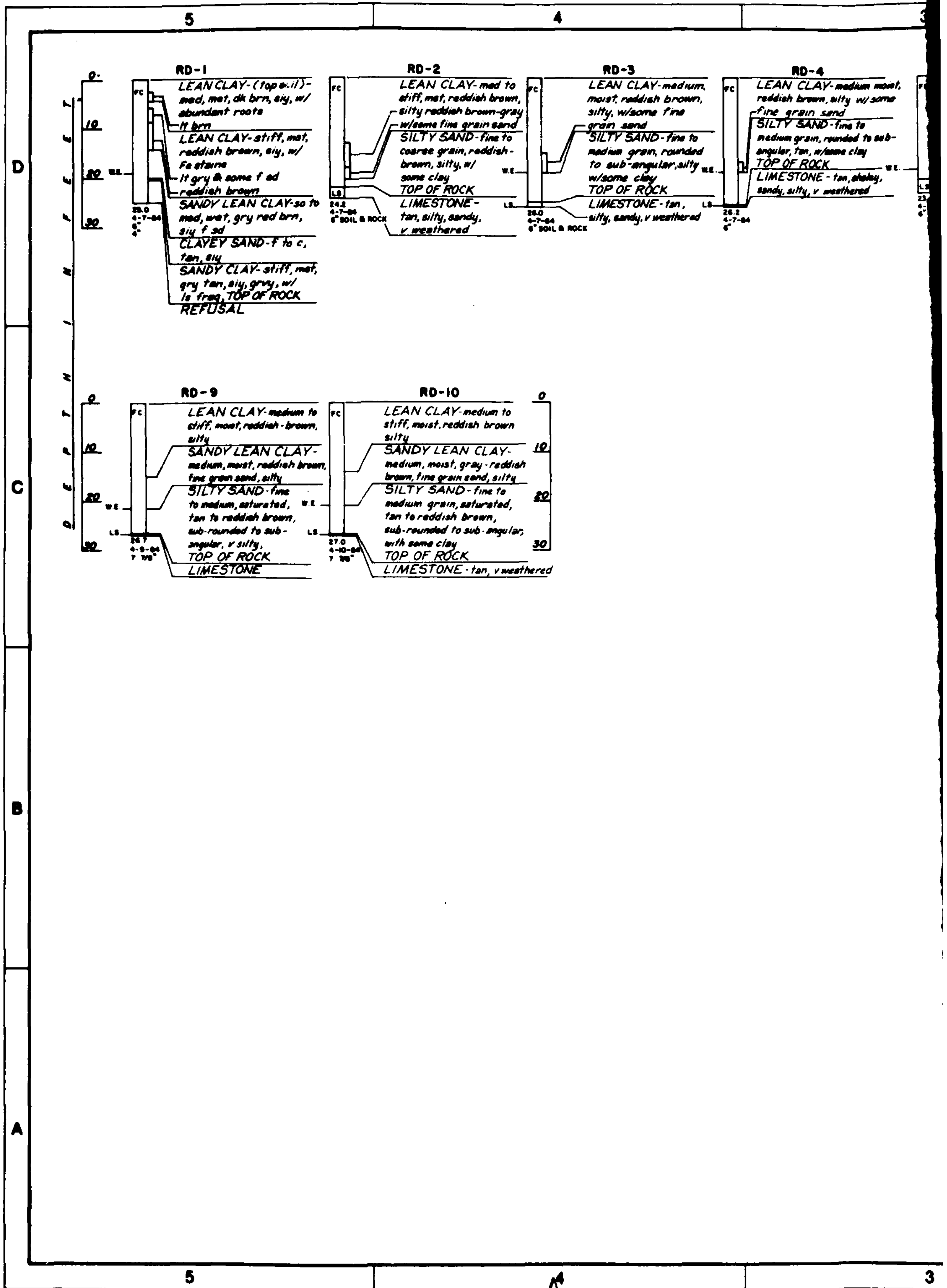
Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

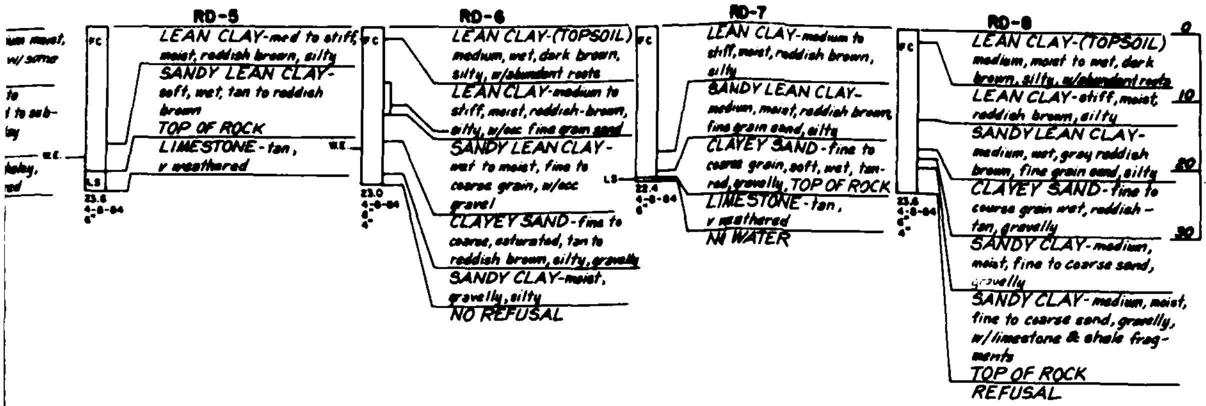
Designed by: J.D.H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by: R.S.T.	LOGS OF EXPLORATIONS
Checked by: J.DH.	Scale: AS SHOWN
Submitted by: F.C.W.	MARCH 1987
	File No. RP-3-1763

3 2 SUPPLEMENT NO 1 PLATE NO. 13

2



5
4
3
D
C
B
A
DEPTH IN FEET
0
10
20
30
RD-1
LEAN CLAY-(top soil)-med, mst, dk brn, siy, w/ abundant roots
lt brn
LEAN CLAY-stiff, mst, reddish brown, siy, w/ Fe stains
lt gry & some f sd
reddish brown
SANDY LEAN CLAY-so to med, wet, gry red brn, siy f sd
CLAYEY SAND-f to c, tan, siy
SANDY CLAY-stiff, mst, gry tan, siy, grvy, w/ ls frag, TOP OF ROCK
REFUSAL
W.E.
FC
23.0
4-7-84
RD-2
LEAN CLAY-med to stiff, mst, reddish brown,
silty reddish brown-gray
w/some fine grain sand
SILTY SAND-fine to coarse grain, reddish-brown, silty, w/ some clay
TOP OF ROCK
LIMESTONE-tan, silty, sandy, v weathered
FC
LS
24.2
4-7-84
6" SOIL & ROCK
RD-3
LEAN CLAY-medium, moist, reddish brown, silty, w/some fine grain sand
SILTY SAND-fine to medium grain, rounded to sub-angular, silty w/some clay
TOP OF ROCK
LIMESTONE-tan, silty, sandy, v weathered
W.E.
LS
FC
26.0
4-7-84
6" SOIL & ROCK
RD-4
LEAN CLAY-medium moist, reddish brown, silty w/some fine grain sand
SILTY SAND-fine to medium grain, rounded to sub-angular, tan, w/some clay
TOP OF ROCK
LIMESTONE-tan, shaley, sandy, silty, v weathered
W.E.
LS
FC
26.2
4-7-84
6"
RD-9
LEAN CLAY-medium to stiff, moist, reddish-brown, silty
SANDY LEAN CLAY-medium, moist, reddish brown, fine grain sand, silty
SILTY SAND-fine to medium, saturated, tan to reddish brown, sub-rounded to sub-angular, v silty,
TOP OF ROCK
LIMESTONE
W.E.
LS
FC
4-9-84
RD-10
LEAN CLAY-medium to stiff, moist, reddish brown silty
SANDY LEAN CLAY-medium, moist, gray-reddish brown, fine grain sand, silty
SILTY SAND-fine to medium grain, saturated, tan to reddish brown, sub-rounded to sub-angular, with some clay
TOP OF ROCK
LIMESTONE-tan, v weathered
W.E.
LS
FC
27.0
4-10-84



Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J. D. H.
Drawn by: P. S. T.
Checked by: J.D.H.
Submitted by: F. C. W.

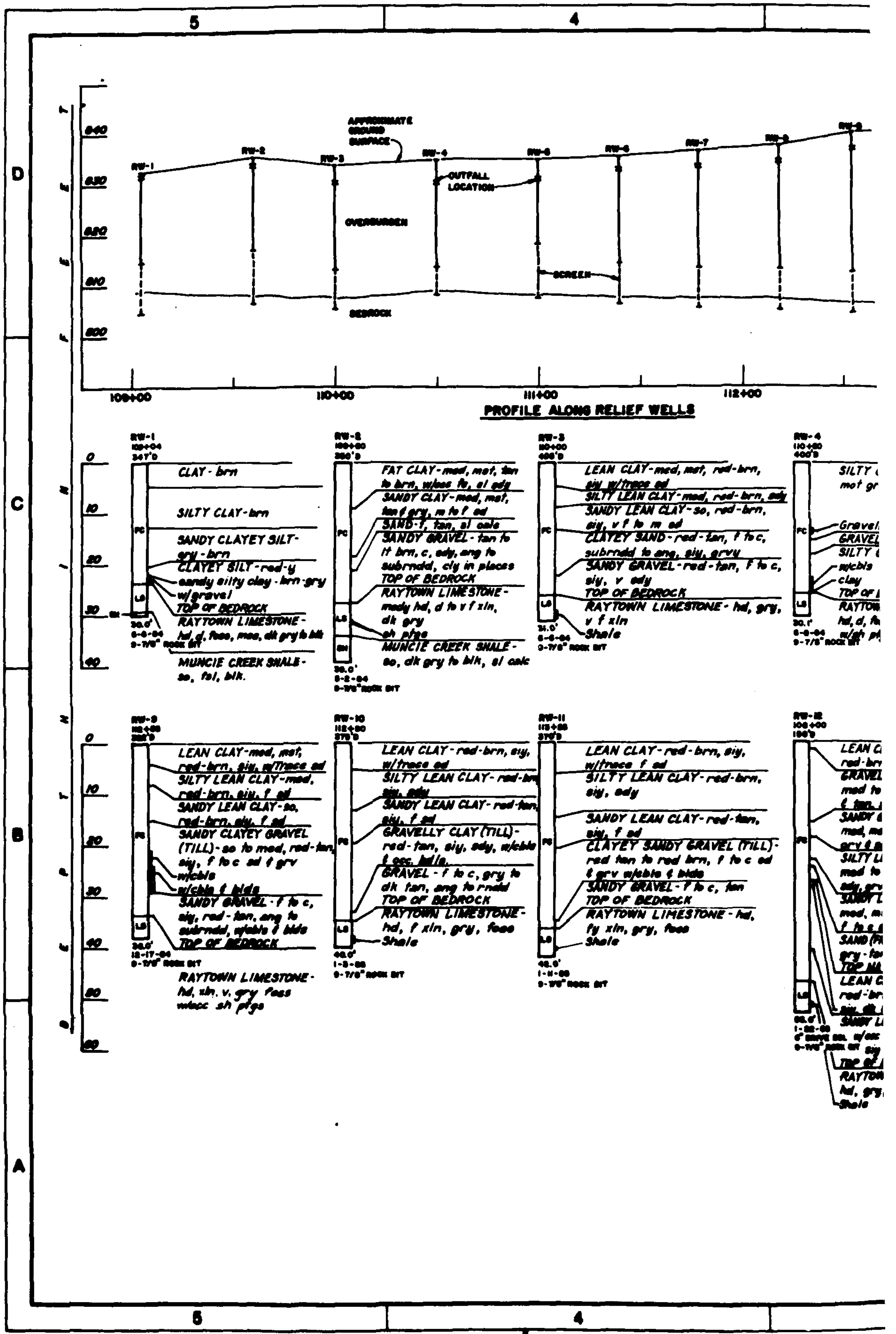
LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT

RELIEF DRAINS
LOGS OF EXPLORATIONS

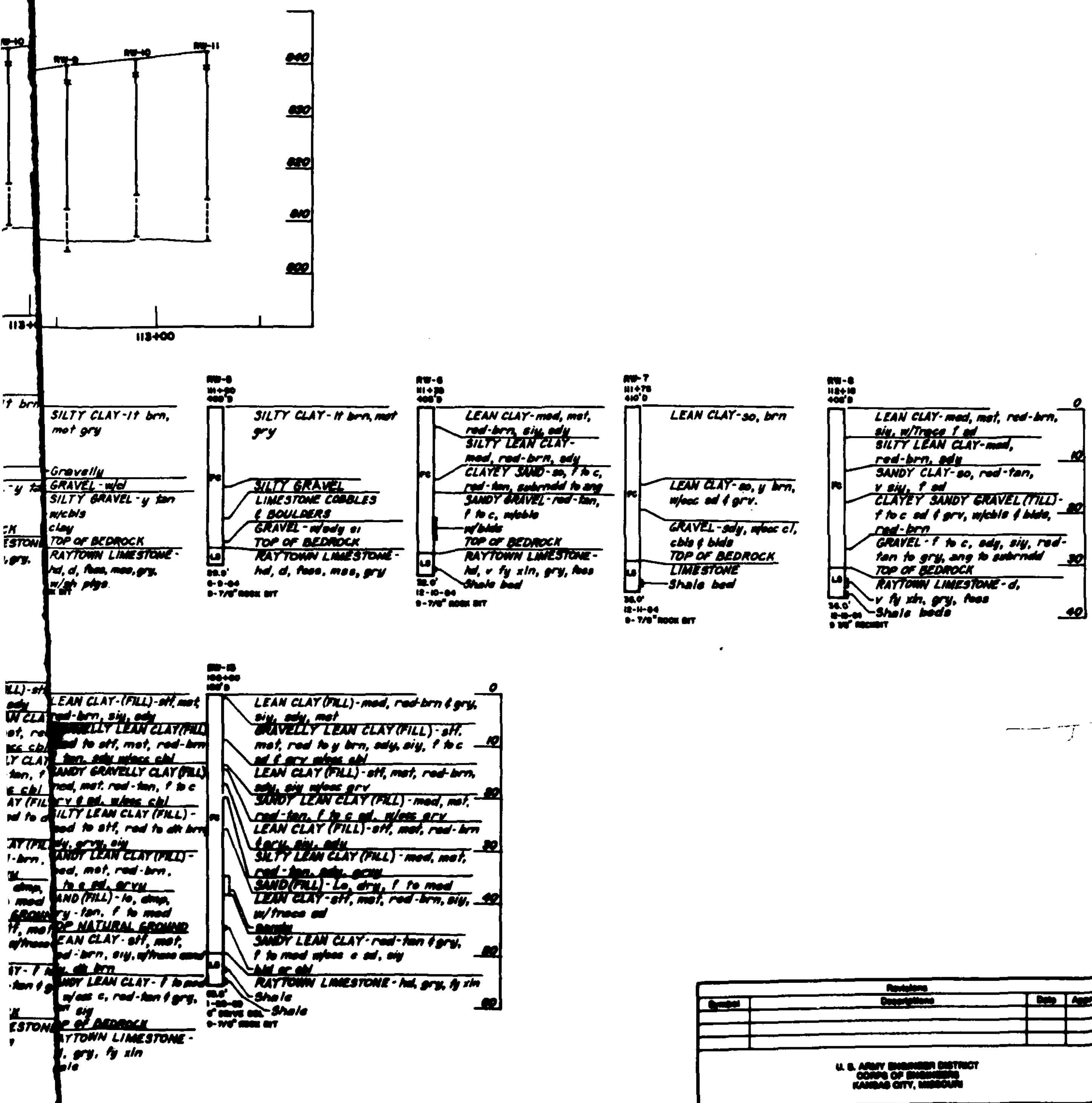
Scale: AS SHOWN
MARCH 1987
File No. RP-3-1764

SUPPLEMENT NO 1 PLATE NO. 14

2



5
4
D
C
B
A
840
830
820
810
800
APPROXIMATE GROUND SURFACE
OUTFALL LOCATION
OVERBURDEN
SCREEN
BEDROCK
RW-1
RW-2
RW-3
RW-4
RW-5
RW-6
RW-7
RW-8
RW-9
109+00
110+00
111+00
112+00
PROFILE ALONG RELIEF WELLS
RW-1
CLAY - brn
SILTY CLAY-brn
SANDY CLAYEY SILT-gry-brn
CLAYEY SILT-red-y
sandy silty clay-brn-gry
w/gravel
TOP OF BEDROCK
RAYTOWN LIMESTONE-
MUNCIE CREEK SHALE-
so, fsl, blk.
RW-2
FAT CLAY-med, mst, tan to brn
SANDY CLAY-med, mst,
SANDY GRAVEL-tan to lt brn, c, sdy, ang to subrndd, cly in places
TOP OF BEDROCK
RAYTOWN LIMESTONE-
mody hd, d to v f xln,
dk gry
sh ptgs
MUNCIE CREEK SHALE-
so, dk gry to blk, sl calc
RW-3
LEAN CLAY-med, mst, red-brn,
SILTY LEAN CLAY-med, red-brn, sdy
SANDY LEAN CLAY-so, red-brn,
siy, v f to m sd
CLAYEY SAND-red-tan, f to c,
subrndd to ang, siy, grvy
SANDY GRAVEL-red-tan, f to c,
siy, v sdy
TOP OF BEDROCK
RAYTOWN LIMESTONE-hd, gry,
v f xln
Shale
RW-4
RW-9
LEAN CLAY-med, mst,
SILTY LEAN CLAY-med,
SANDY LEAN CLAY-so,
SANDY CLAYEY GRAVEL
(TILL)-so to med, red-tan,
siy, f to c sd & grv
w/cble
w/cble & blds
SANDY GRAVEL-f to c,
siy, red-tan, ang to subrndd, w/cble & blds
TOP OF BEDROCK
RAYTOWN LIMESTONE-
hd, xln, v. gry foss
w/occ sh ptgs
RW-10
LEAN CLAY-red-brn, siy,
w/trace sd
SILTY LEAN CLAY-red-brn
SANDY LEAN CLAY-red-tan,
siy, f sd
GRAVELLY CLAY (TILL)-
red-tan, siy, sdy, w/cble
& occ. blds
GRAVEL-f to c, gry to
dk tan, ang to rndd
TOP OF BEDROCK
RAYTOWN LIMESTONE-
hd, f xln, gry, foss
Shale
RW-11
LEAN CLAY-red-brn, siy,
w/trace f sd
SILTY LEAN CLAY-red-brn,
siy, sdy
SANDY LEAN CLAY-red-tan,
siy, f sd
CLAYEY SANDY GRAVEL (TILL)-
red tan to red brn, f to c sd
& grv w/cble & blds
SANDY GRAVEL-f to c, tan
TOP OF BEDROCK
RAYTOWN LIMESTONE-hd,
fy xln, gry, foss
Shale
RW-12



Revisions			
Symbol	Descriptions	Date	Approved

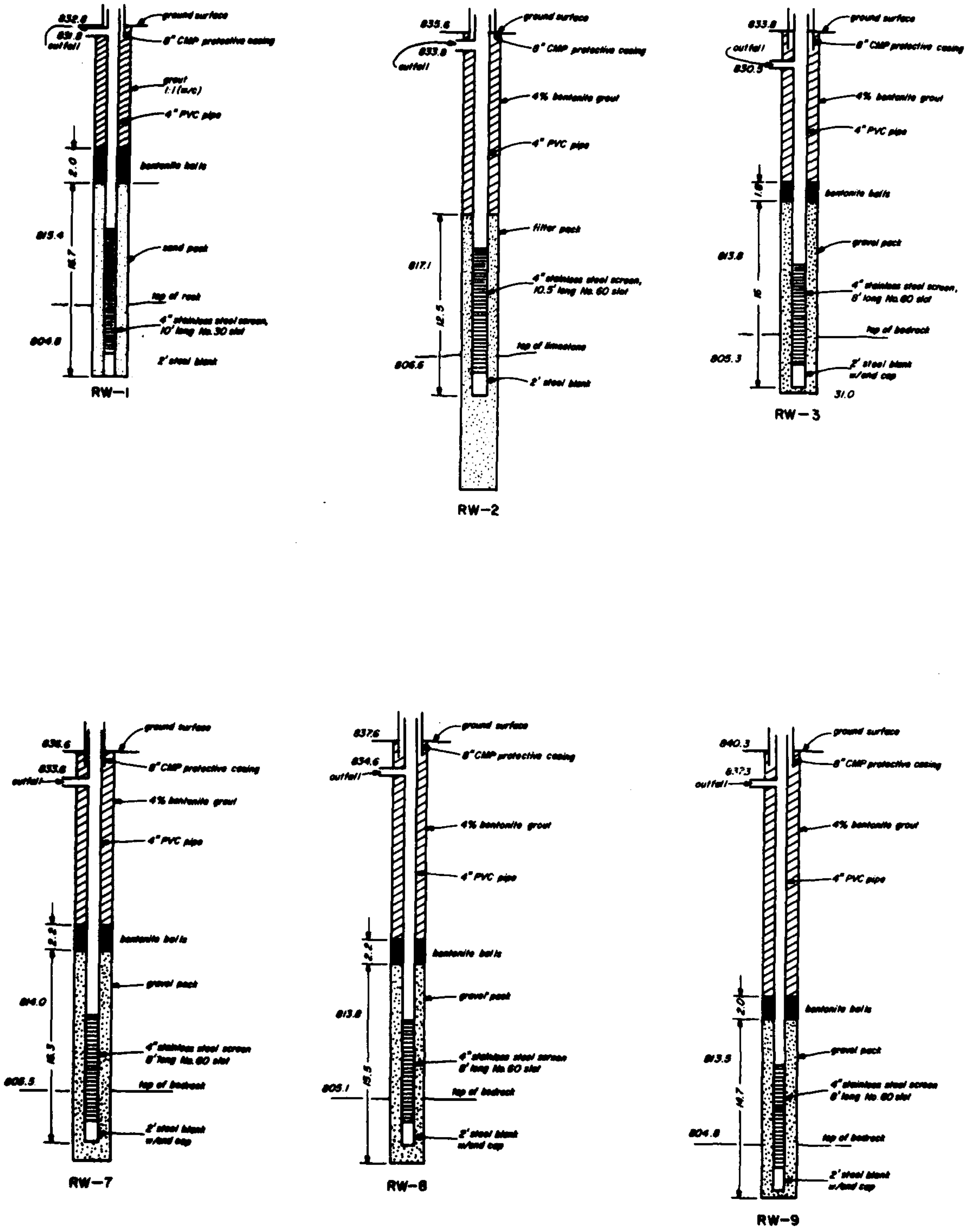
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J.B.H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT PROFILE ALONG RELIEF WELLS AND LOGS OF EXPLORATIONS
Drawn by: R.S.T.	
Checked by: J.B.H.	Scale: AS SHOWN
Submitted by: R.C.W.	MARCH 1987
	File No. RP-3-1765

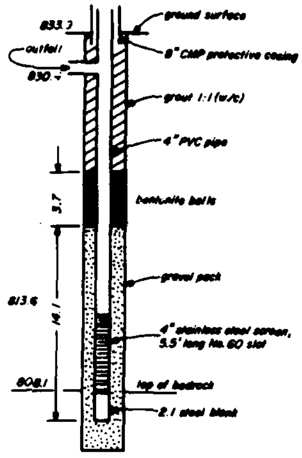
SUPPLEMENT NO. 1 PLATE NO. 15

1

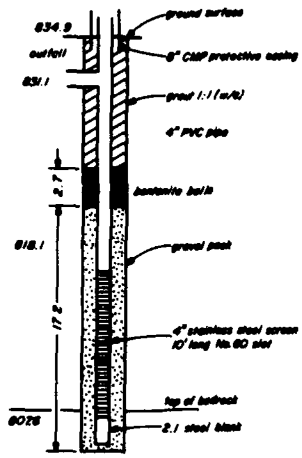
2



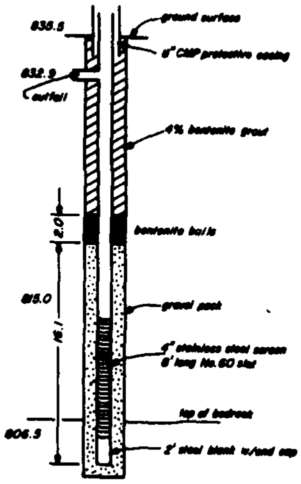
RW-1
ground surface
8" CMP protective casing
outfall
grout 1:1 (w/c)
4" PVC pipe
bentonite balls
sand pack
top of rock
4" stainless steel screen, 10' long No. 30 slot
2' steel blank
2.0
815.4
18.7
804.8
RW-2
ground surface
8" CMP protective casing
outfall
4% bentonite grout
4" PVC pipe
filter pack
4" stainless steel screen, 10.5' long No. 60 slot
top of limestone
2' steel blank
835.6
833.8
817.1
12.5
806.6
RW-3
ground surface
8" CMP protective casing
outfall
4% bentonite grout
4" PVC pipe
bentonite balls
gravel pack
4" stainless steel screen, 8' long No. 60 slot
top of bedrock
2' steel blank w/end cap
833.8
830.5
1.8
813.8
16
805.3
31.0
RW-7
ground surface
8" CMP protective casing
outfall
4% bentonite grout
4" PVC pipe
bentonite balls
gravel pack
4" stainless steel screen 8' long No. 60 slot
top of bedrock
2' steel blank w/end cap
836.6
833.6
2.2
814.0
808.5
RW-8
ground surface
8" CMP protective casing
outfall
4% bentonite grout
4" PVC pipe
bentonite balls
gravel pack
4" stainless steel screen 8' long No. 60 slot
top of bedrock
2' steel blank w/end cap
837.6
834.6
2.2
813.8
15.5
805.1
RW-9
ground surface
8" CMP protective casing
outfall
4% bentonite grout
4" PVC pipe
bentonite balls
gravel pack
4" stainless steel screen 8' long No. 60 slot
top of bedrock
2' steel blank w/end cap
840.3
837.3
2.0
813.5
14.7
804.8



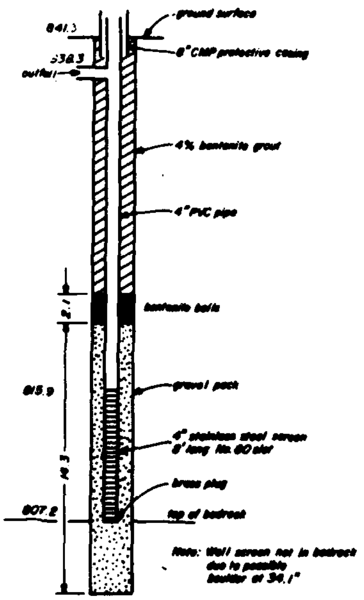
RW-4



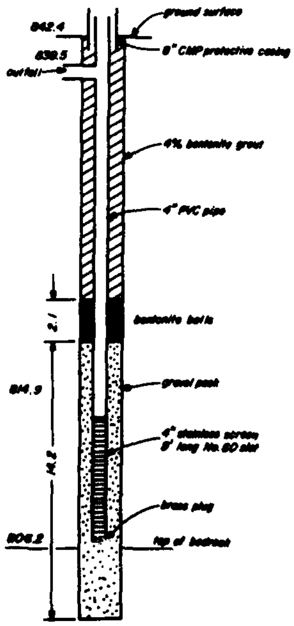
RW-5



RW-6



RW-10



RW-11

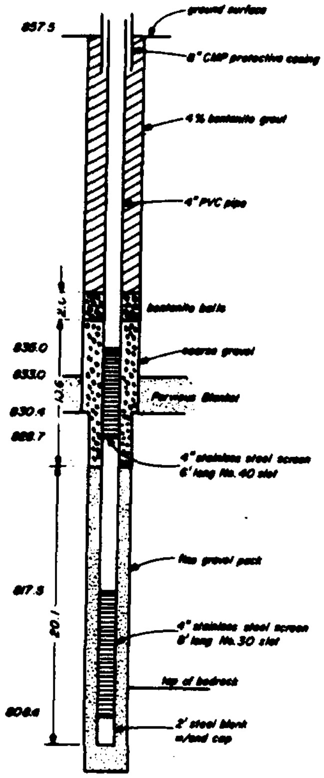
Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

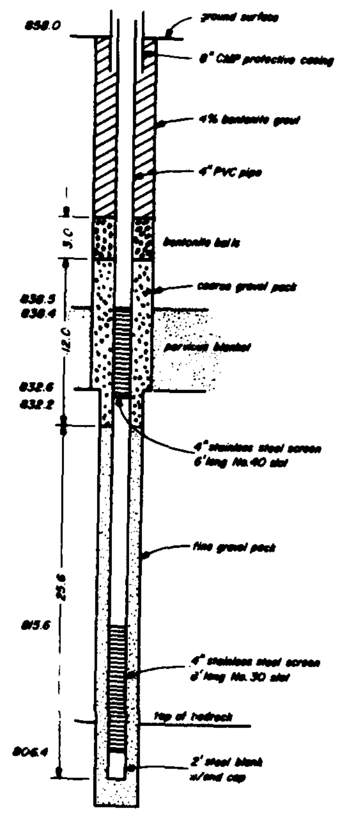
Designed by: J.B.H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by: J.W.T.	INSTALLATION DETAILS OF RW-1 THRU RW-11
Checked by: J.B.H.	Scale: AS SHOWN
Submitted by: F.C.W.	MARCH 1987
	File No. RP-3-1766

2

RW-12 AND RW-13
WELLS INTERCEPTING PERVIOUS BLANKET



RW-12

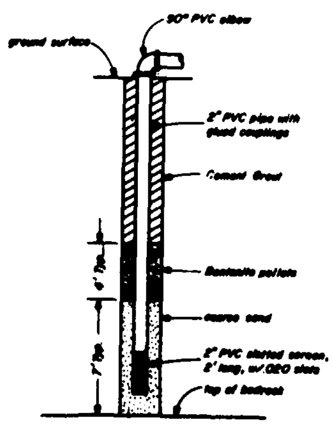


RW-13

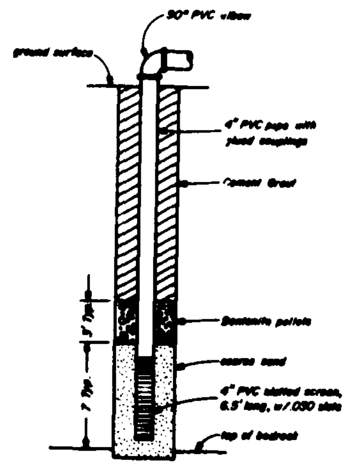
863.2

813.2

803.2

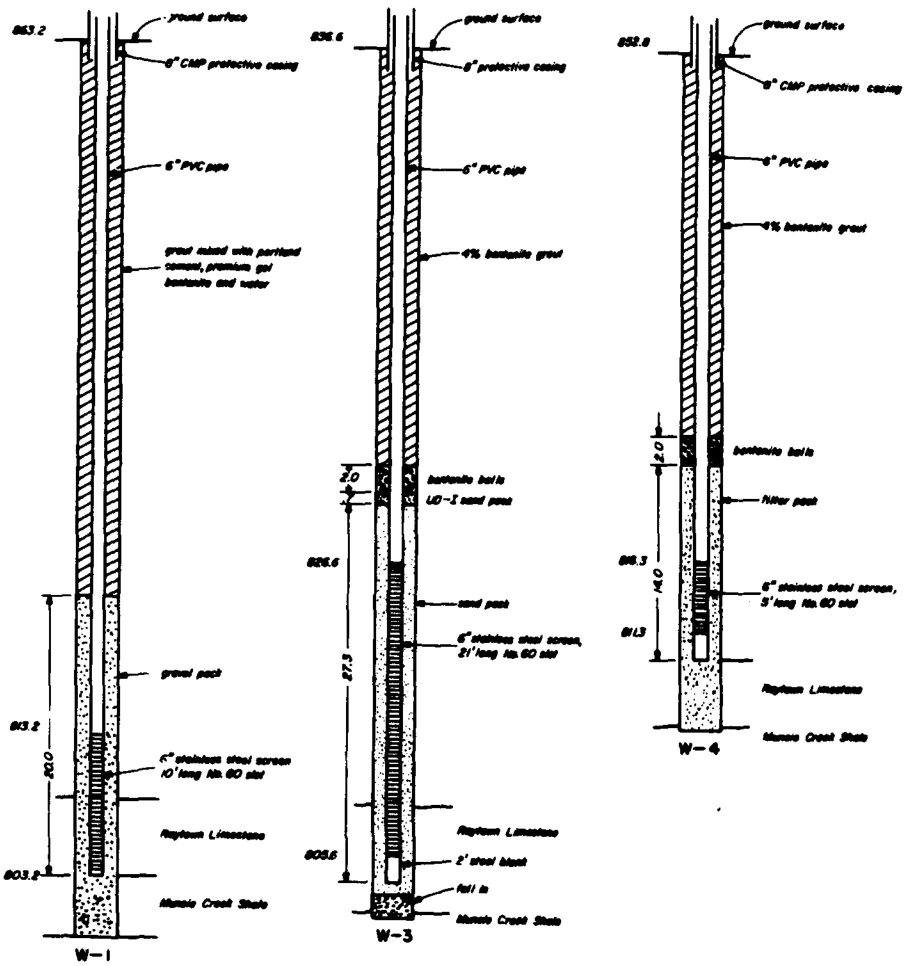


TYPICAL 2-INCH RELIEF DRAIN



TYPICAL 4-INCH RELIEF DRAIN

W-1, W-3, AND W-4
PUMPED WELLS



U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

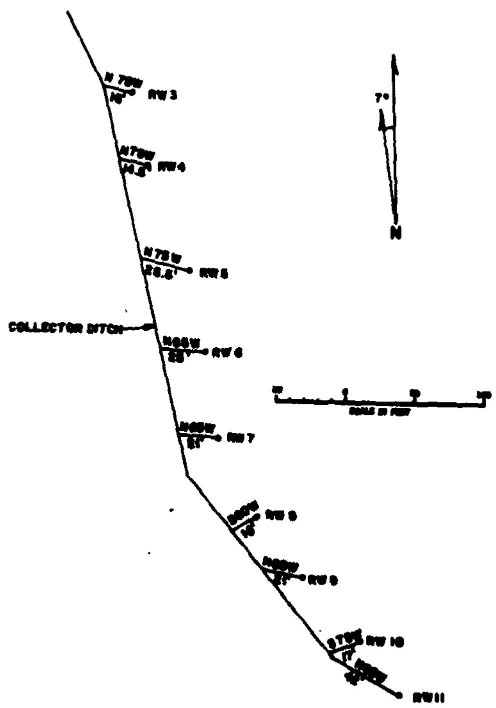
LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT
INSTALLATION DETAILS OF RW-[illegible]
& RW-[illegible], PUMPED WELLS AND
2-INCH AND 4-INCH RELIEF DRAINS

J.G.H.
P.S.T.
J.D.H.
F.C.W.

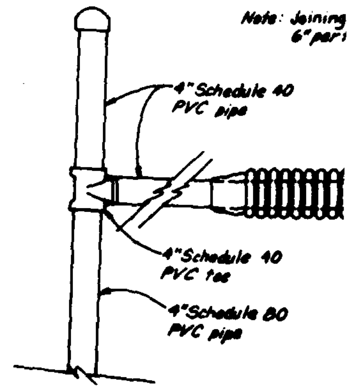
AS SHOWN

MARCH 1967

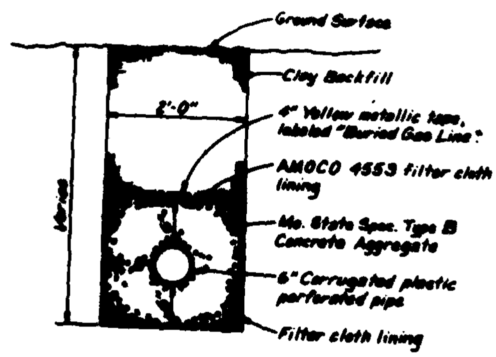
RP-3-1767



SMITHVILLE RELIEF WELLS AND MAGNETIC BEARING AND LENGTH OF RELIEF WELL LATERALS

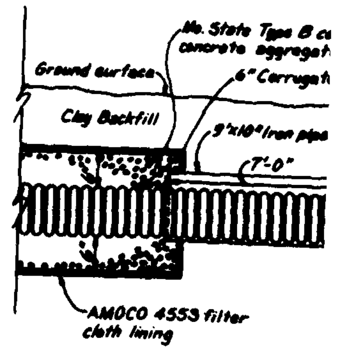


DETAILS OF RELIEF
COLLECTOR PIP

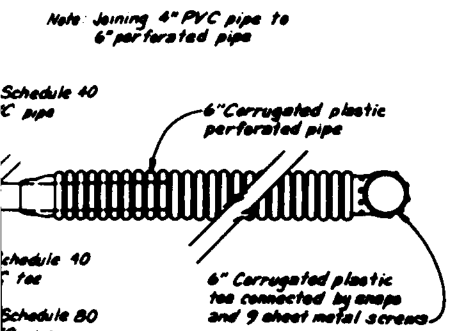


DETAILS OF RELIEF WELL COLLECTOR DITCH

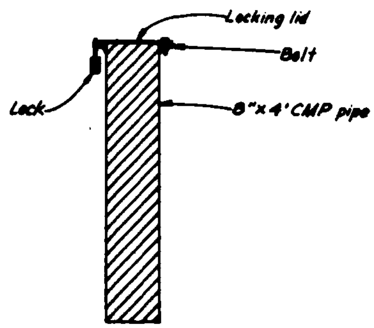
N.T.S.



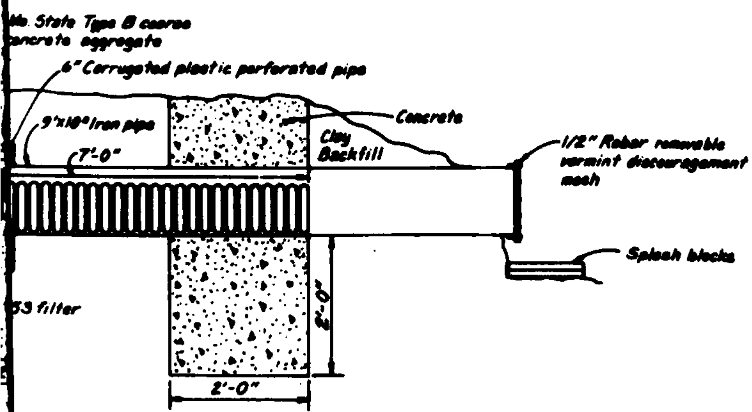
DETAIL OF
SYSTEM SHOWIN



AILS OF RELIEF WELL, LATERALS, AND COLLECTOR PIPE CONNECTIONS
N.T.S.



DETAILS OF RELIEF WELL COVER
N.T.S.



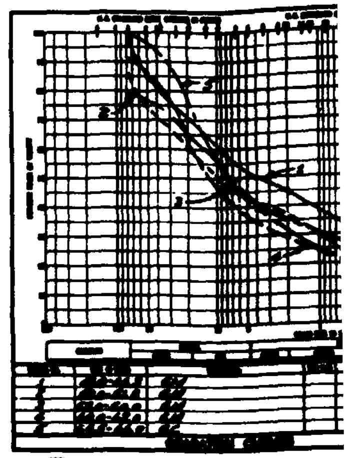
DETAIL OF TERMINATION OF COLLECTOR SYSTEM SHOWING 6" TO 10" PIPE CONNECTION
N.T.S.

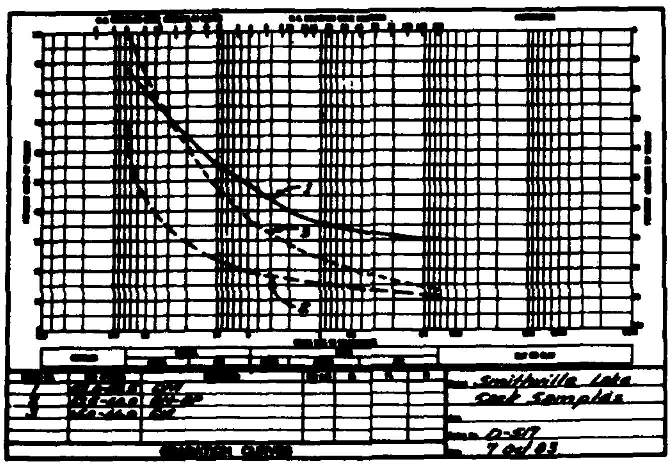
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J.G.H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by: V.A.S.	DETAILS OF COLLECTOR DITCH
Checked by: J.G.H.	Scale: AS SHOWN
Submitted by: E.G.W.	MARCH 1967
	File No. RP-3-1768

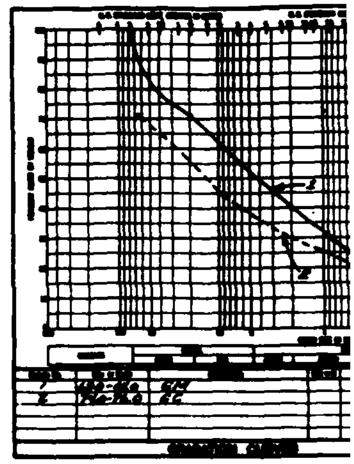
SUPPLEMENT NO 1 PLATE NO. 18

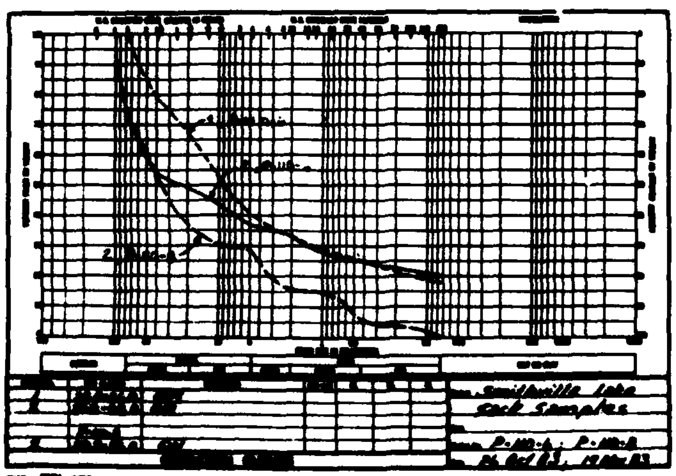
Smithville Lake
Sack Samples
D-53
21 Sept 83



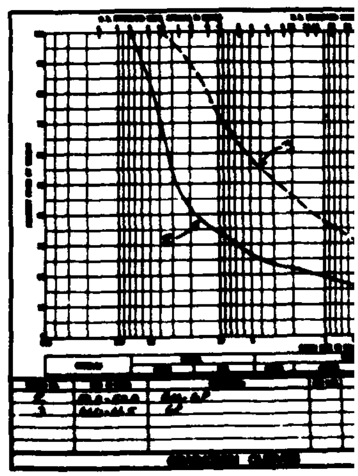


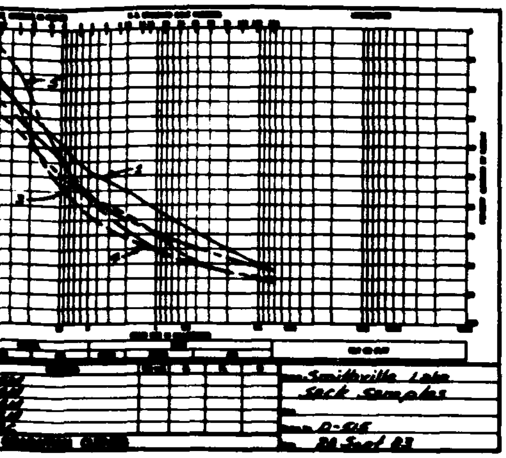
Smithville Lake
Sack Samples
D-57
7 Oct 83

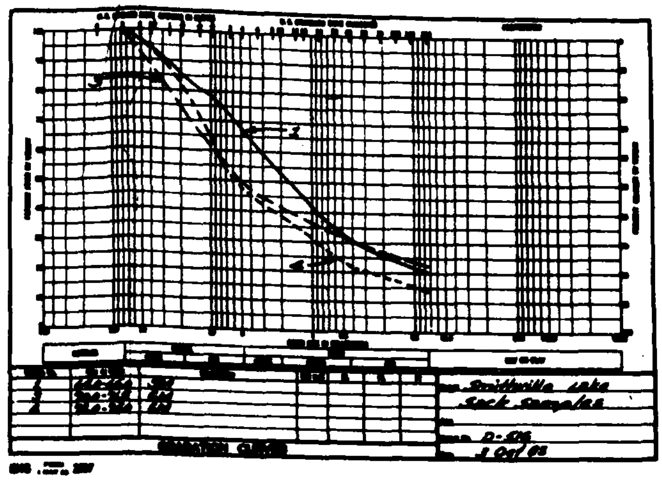


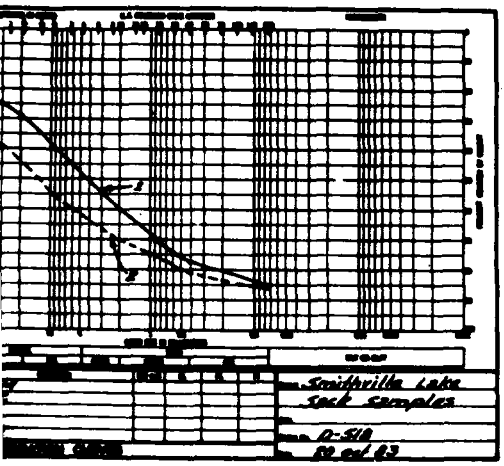


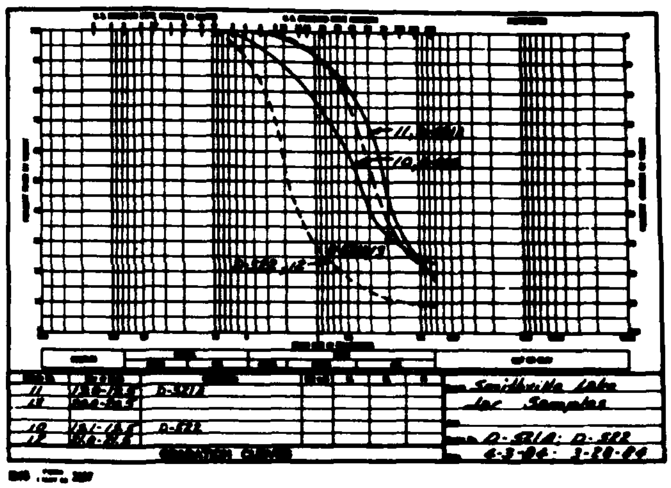
Smithville Lake
Sack Samples

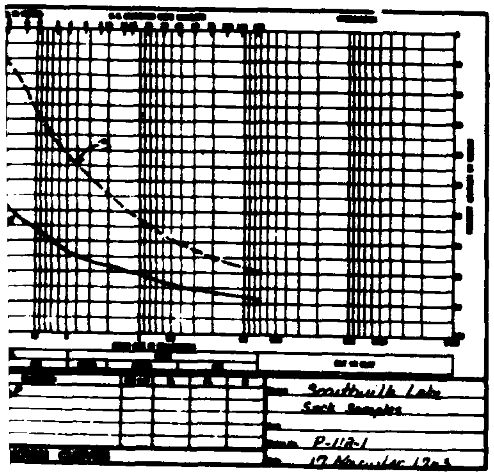












Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by	J.M.M.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by	P.S.T.	GRADATION CURVES LEFT ABUTMENT BORINGS
Checked by	J.D.H.	Scale AS SHOWN
Submitted by	F.C.W.	MARCH 1987 — File No. RP-3-1769

2

5 4 3

D

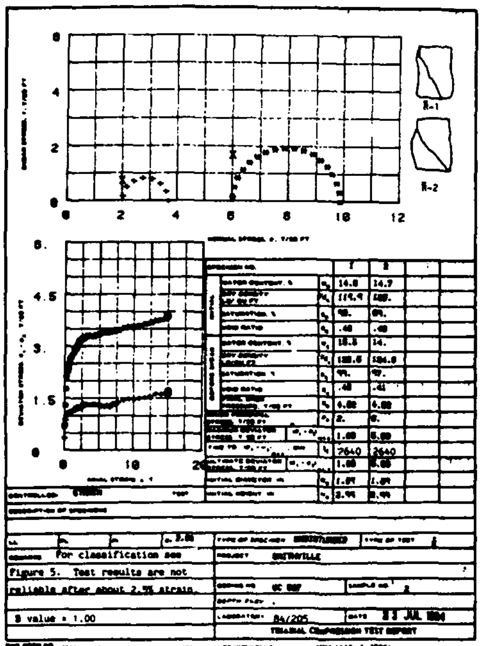
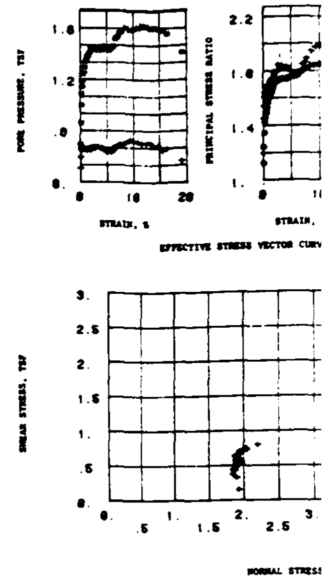


Figure 3



REMARKS: COMPUTER PRINT-OUT
SYMBOLS SAME AS FORM 2089
R TRIAXIAL TEST: PORE PRESSURE
MONITORED DURING SHEAR

PROJEC
BORING
DEPTH/
MRD LA

MRD Form 756, 1 May 80

C

B

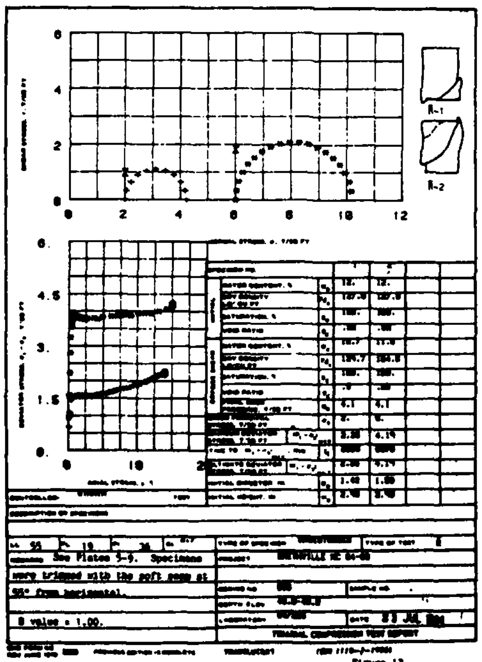
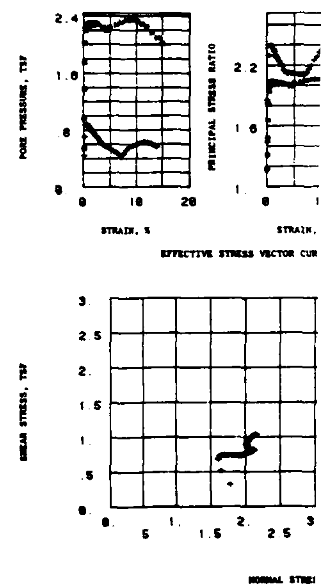


Figure 12

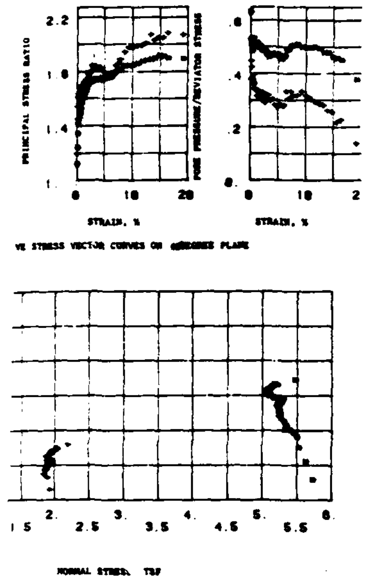


REMARKS: COMPUTER PRINT-OUT
SYMBOLS SAME AS FORM 2089
R TRIAXIAL TEST: PORE PRESSURE
MONITORED DURING SHEAR

MRD Form 756, 1 May 80

A

5 4 3



PROJECT: SMITHVILLE
BORING NO: UC 527 SAMPLE NO:
DEPTH/ELEV:
KCD LAB NO: 84/205 DATE: 13 JUL 1984

TRIAXIAL COMPRESSION TEST REPORT

FIGURE: 4

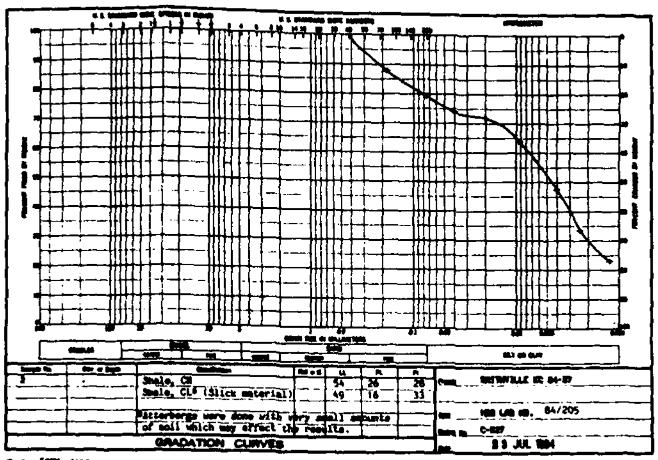
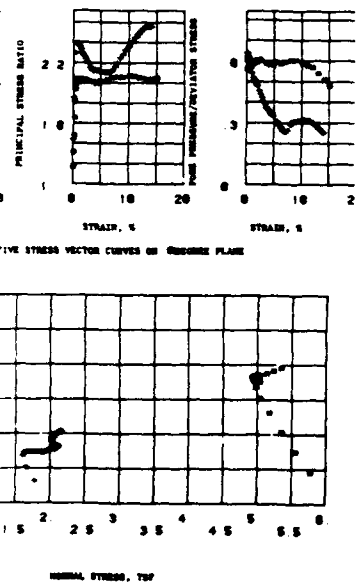


FIGURE 5



PROJECT: SMITHVILLE
BORING NO: SAMPLE NO:
DEPTH/ELEV:
KCD LAB NO: DATE: JUL 1984

TRIAXIAL COMPRESSION TEST REPORT

FIGURE: 13

Revisions			
Symbol	Descriptions	Date	Approved

U S ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY MISSOURI

Designed by: J.D.H.
Drawn by: J.W.P.
Checked by: J.D.H.
Submitted by: F.C.W.

US Army Corps of Engineers

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT
TEST DATA
LEFT ABUTMENT, R TESTS
BORINGS UC-527 (P-109-0, C-533 (P-L2-2)

Scale: AS SHOWN

MARCH 1987

File No. RP-3-1770

SUPPLEMENT NO. 1 PLATE NO. 20

5 4

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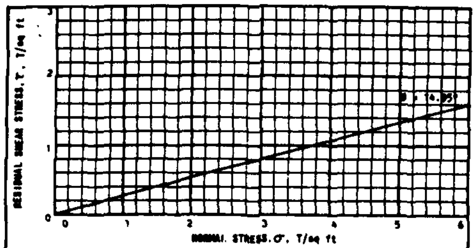


TEST NO.					RESIDUAL SHEAR STRESS, T/sq ft	1.60		
INITIAL	WATER CONTENT %	w_o			τ/σ AT RESIDUAL	.27		
	VOID RATIO	e_o			TIME TO RESIDUAL, days	8		
	SATURATION %	S_o			DISPLACEMENT TO RESIDUAL, inches	2.43		
	DRY DENSITY, lb/cu ft	γ_d			FINAL SHEAR STRESS, T/sq ft	2.28		
SPECIFIC GRAVITY		G_s			FINAL DISPLACEMENT, inches	3.74		
LIQUID LIMIT		LL	44		LENGTH OF TEST, days	11		
PLASTIC LIMIT		PL	16		MATERIAL EXTRUDED, gms			
WATER CONTENT %		w_f			MATERIAL EXTRUDED, %			
NORMAL STRESS, T/sq ft		σ	6.0		SHEAR STRENGTH PARAMETERS: φ MAXIMUM			
MAXIMUM SHEAR STRESS, T/sq ft		τ_{max}	3.68		φ RESIDUAL	14.95		
DISPLACEMENT TO MAXIMUM, inches			.06		COHESION T/sq ft			

SPECIMEN DIMENSIONS 3.01 IN. SQUARE IN. THICK	MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
CONTROLLED ☒ STRAIN ☐ STRESS	PROJECT Smithville LAB. NO. 84/205
TYPE OF SPECIMEN Undisturbed, Intact	AREA
CLASSIFICATION	BORING NO. C-527 SAMPLE NO. 3
	DEPTH ELEVATION 43.8' DATE 23 JUL 1984
	RESIDUAL DIRECT SHEAR TEST REPORT

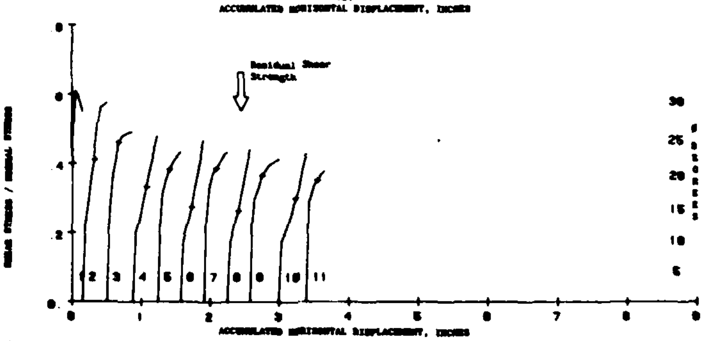
MRD FORM 1 NOV 68 760

Figure 6

C

B

SHEAR STRESS/NORMAL STRESS VS. ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



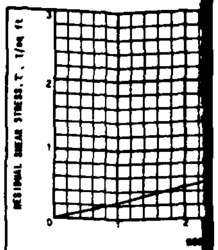
PULL NO:	1	2	3	4	5	6	7	8	9	10	11
IN./DAY:	.19	.33	.36	.32	.4	.33	.33	.33	.33	.16	.04

SPECIMEN TYPE: SHEAR, Undisturbed
NORMAL STRESS, T/SF: 6.0
REMARKS: See Plate 3.

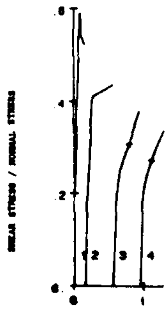
PROJECT: Smithville
BORING NO: C-527 SAMPLE NO: 3
DEPTH: 43.8'
MRD LAB NO: 84/205 DATE: 23 JUL 1984
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 7

MRD Form 760, 1 May 80 (Replaces 1 Nov 68 edition)

A



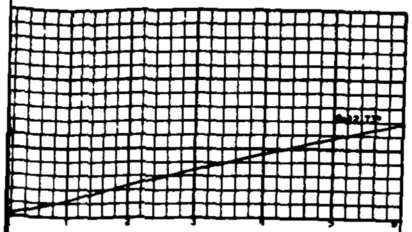
TEST NO.; LIQUID LIMIT LL 45; PLASTIC LIMIT PL 15; NORMAL STRESS σ 6.0; MAXIMUM SHEAR STRESS τ_{max} 3.54; DISPLACEMENT TO MAXIMUM .10
SPECIMEN DIMENSIONS 3.004 IN. SQUARE IN. THICK
CONTROLLED ☒ STRAIN ☐ STRESS
TYPE OF SPECIMEN Undisturbed, Intact
CLASSIFICATION See Plate 2
MRD FORM 1 NOV 68 760



PULL NO: 1 2
IN./DAY: .33 .42
SPECIMEN TYPE:
NORMAL STRESS, T/SF: 6.0
REMARKS:
MRD Form 760, 1 May 80

5 4

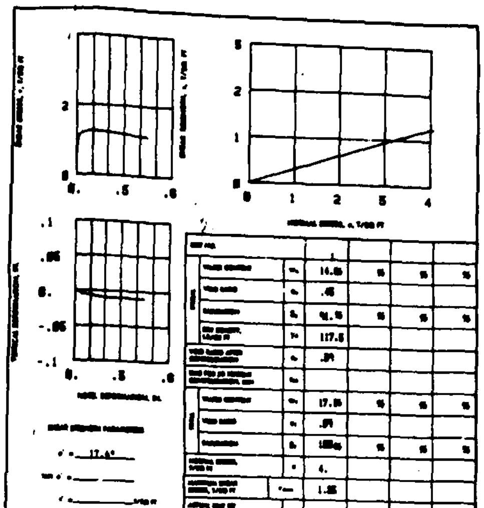
3 2 1



				RESIDUAL SHEAR STRESS, T/sq ft	1.36		
[illegible]	%			τ/σ AT RESIDUAL	.22		
[illegible]	%			TIME TO RESIDUAL, days	7		
[illegible]	%			DISPLACEMENT TO RESIDUAL, inches	2.20		
[illegible]	%			FINAL SHEAR STRESS, T/sq ft	1.43		
[illegible]	%			FINAL DISPLACEMENT, inches	3.21		
	LL	45		LENGTH OF TEST, days	10		
	PL	15		MATERIAL EXTRUDED, gms	7.19		
				MATERIAL EXTRUDED, %			
T/sq ft	σ	6.0		SHEAR STRENGTH PARAMETERS: φ MAXIMUM			
	τ_{max}	1.54		φ RESIDUAL	12.73		
TO MAXIMUM		.10		COHESION T/sq ft			

[illegible] SQUARE [illegible] THICK	MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
STRAIN / STRESS	PROJECT Smithville Dam LAB. NO. 84/205
Undisturbed, Intact	AREA
See Plate 2	BORING NO. C-527 SAMPLE NO. 2
	DEPTH ELEVATION 41.3 DATE 22 JUN 1984
	RESIDUAL DIRECT SHEAR TEST REPORT

Figure 7



SHEAR STRENGTH PARAMETERS

φ = 17.6°

TYPE OF SPECIMEN: UNDISTURBED

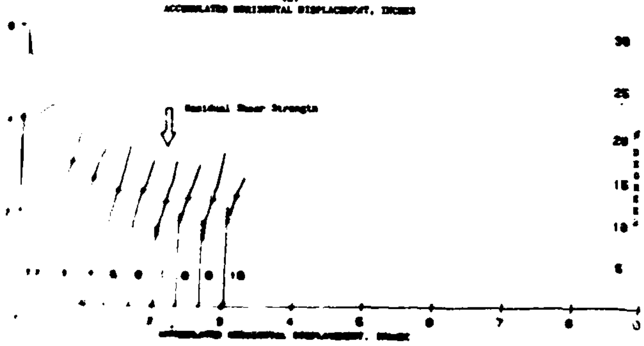
PROJECT SMITHVILLE DAM

BORING NO. C 532 SAMPLE NO. 1

DIRECT SHEAR TEST REPORT

Figure 1

SHEAR STRESS/NORMAL STRESS VS. ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



PROJECT: Smithville

BORING NO.: C-527 SAMPLE NO.: 2

RESIDUAL DIRECT SHEAR TEST REPORT

Figure 6

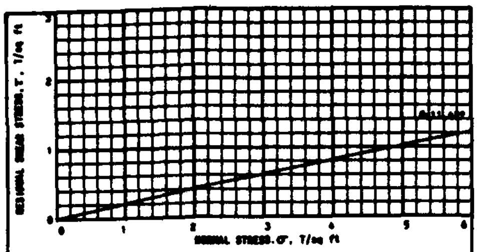
Symbol	Descriptions	Date	Approved
	Revisions		

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by J.D.H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT TEST DATA-LEFT ABUTMENT RESIDUAL S TEST, BORING C-527 (P-109-6) S TEST, BORING C-532 (P-110-10)
Drawn by J.W.	
Checked by L.B.H.	Scale AS SHOWN
Submitted by FCW	MARCH 1987
	File No. RP-3-1771

5 4

D



TEST NO.					RESIDUAL SHEAR STRESS, T/sq ft	1.21		
INITIAL	WATER CONTENT	%			τ/σ AT RESIDUAL	.20		
	VOID RATIO	e_o			TIME TO RESIDUAL, days	7		
	SATURATION	S_o			DISPLACEMENT TO RESIDUAL, inches	1.99		
	DRY DENSITY, T/cu ft	γ_d			FINAL SHEAR STRESS, T/sq ft	1.53		
SPECIFIC GRAVITY		G_s			FINAL DISPLACEMENT, inches	2.20		
LIQUID LIMIT		LL			LENGTH OF TEST, days	7		
PLASTIC LIMIT		PL			MATERIAL EXTRUDED, gm	-		
WATER CONTENT		w_f			MATERIAL EXTRUDED, %	-		
NORMAL STRESS, T/sq ft		σ	6.0		SHEAR STRENGTH PARAMETERS: φ MAXIMUM	-		
MAXIMUM SHEAR STRESS, T/sq ft		τ_{max}	2.54		φ RESIDUAL	11.42		
DISPLACEMENT TO MAXIMUM, inches			.06		COHESION T/sq ft	-		

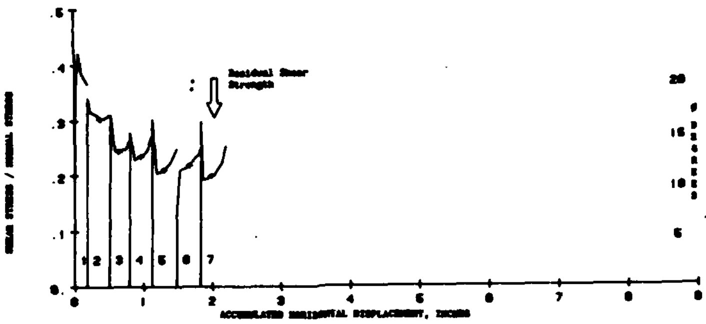
SPECIMEN DIMENSIONS 3.01 IN. SQUARE IN. THICK	MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
CONTROLLED ☒ STRAIN ☐ STRESS	PROJECT Smithville LAB. NO. 84/205
TYPE OF SPECIMEN Undisturbed, Precut	AREA
CLASSIFICATION	BORING NO. C-526 SAMPLE NO. 4
	DEPTH ELEVATION 47.2' DATE [illegible] JUL [illegible]
	RESIDUAL DIRECT SHEAR TEST REPORT

MRD FORM 1 NOV 68 760

Figure 1

C

SHEAR STRESS/NORMAL STRESS
VS.
ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



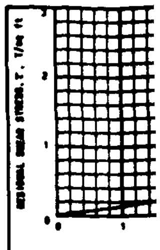
B

PULL NO:	1	2	3	4	5	6	7
IN./DAY:	.10	.[illegible]	.14	.[illegible]	.[illegible]	.[illegible]	.[illegible]

SPECIMEN TYPE: UNDISTURBED, PRECUT
NORMAL STRESS, T/SF: 6.0
REMARKS:

PROJECT: Smithville
BORING NO: [illegible] SAMPLE NO: 4
DEPTH: 47.2
MRD LAB NO: 84/205 DATE: [illegible] JUL [illegible]
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 2

MRD Form 760, 1 May 80 (Replaces 1 Nov 68 edition)

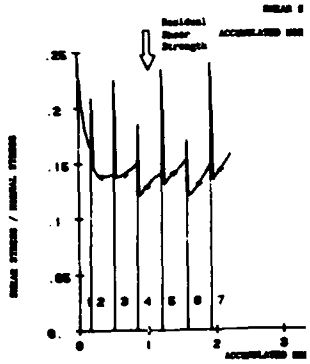


TEST NO.			
INITIAL	WATER CONTENT	%	
	VOID RATIO	e_o	
	SATURATION	S_o	
	DRY DENSITY, T/cu ft	γ_d	
SPECIFIC GRAVITY		G_s	
LIQUID LIMIT		LL	97
PLASTIC LIMIT		PL	18
WATER CONTENT		w_f	
NORMAL STRESS, T/sq ft		σ	6.0
MAXIMUM SHEAR STRESS, T/sq ft		τ_{max}	1.3
DISPLACEMENT TO MAXIMUM, inches			.02

SPECIMEN DIMENSIONS 3.00 IN. SQUARE
CONTROLLED ☒ STRAIN ☐ ST
TYPE OF SPECIMEN Undisturbed,
CLASSIFICATION

MRD FORM 1 NOV 68 760

SHEAR S
ACCUMULATED HOR



PULL NO:	1	2	3	4	5	6	7
IN./DAY:	.[illegible]	.[illegible]	.[illegible]	.[illegible]	.[illegible]	.11	.[illegible]

SPECIMEN TYPE: PRECUT
NORMAL STRESS, T/SF: 6.0
REMARKS:

MRD Form 760, 1 May 80 (Replaces 1 Nov 68 edition)

A

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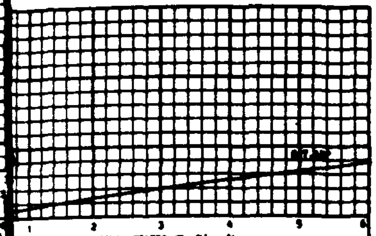


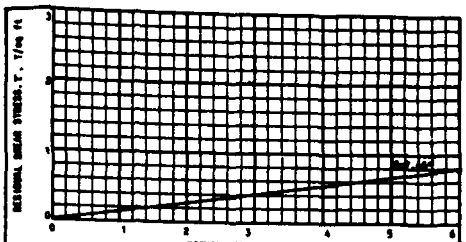
RESIDUAL SHEAR STRESS, T/sq ft	.98		
τ/σ AT RESIDUAL	.13		
TIME TO RESIDUAL, days	4		
DISPLACEMENT TO RESIDUAL, inches	1.07		
FINAL SHEAR STRESS, T/sq ft	.87		
FINAL DISPLACEMENT, inches	2.08		
LENGTH OF TEST, days	7		
MATERIAL EXTRUDED, gms			
MATERIAL EXTRUDED, %			
SHEAR STRENGTH PARAMETERS ϕ MAXIMUM			
ϕ RESIDUAL	7.42		
COHESION T/sq ft			

LL 57; PL 18; σ 6.0; τ max 1.38; .02

MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
PROJECT Smithville LAB. NO. 84/205
BORING NO. C-526 SAMPLE NO. 2
ELEVATION 43.3 DATE 88 JUN 884

RESIDUAL DIRECT SHEAR TEST REPORT

Figure 3



TEST NO.					RESIDUAL SHEAR STRESS, T/sq ft	.79		
INITIAL	WATER CONTENT %	%			τ/σ AT RESIDUAL	.13		
	VOID RATIO	%			TIME TO RESIDUAL, days	6		
	SATURATION %	%			DISPLACEMENT TO RESIDUAL, inches	1.83		
	DRY DENSITY, T/cu ft	γd			FINAL SHEAR STRESS, T/sq ft	.85		
SPECIFIC GRAVITY		G			FINAL DISPLACEMENT, inches	2.87		
LIQUID LIMIT		LL	57		LENGTH OF TEST, days	9		
PLASTIC LIMIT		PL	18		MATERIAL EXTRUDED, gms			
WATER CONTENT %		wf			MATERIAL EXTRUDED, %			
NORMAL STRESS, T/sq ft		σ	6.0		SHEAR STRENGTH PARAMETERS ϕ MAXIMUM			
MAXIMUM SHEAR STRESS, T/sq ft		τ max	1.78		ϕ RESIDUAL	7.46		
DISPLACEMENT TO MAXIMUM, inches			.02		COHESION T/sq ft			

SPECIMEN DIMENSIONS 1.906 IN. SQUARE IN. THICK	MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
CONTROLLED ☒ STRAIN ☐ STRESS	PROJECT Smithville LAB. NO. 84/205
TYPE OF SPECIMEN Undisturbed, Precut	AREA
CLASSIFICATION	BORING NO. C-526 SAMPLE NO. 2
	DEPTH ELEVATION 43.3 DATE 88 JUN 884
	RESIDUAL DIRECT SHEAR TEST REPORT

MRD FORM 1 NOV 68 760

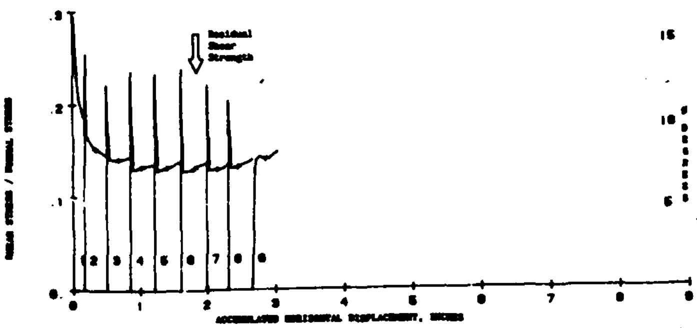
Figure 5

SHEAR STRESS/NORMAL STRESS
VS.
ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES

PULL NO.: ... 7
IN./DAY: ... .01

PROJECT: Smithville
BORING NO.: C-526 SAMPLE NO.: 2
DEPTH:
MRD LAB NO.: 84/205 DATE: 88 JUN 884
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 4

SHEAR STRESS/NORMAL STRESS
VS.
ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



PULL NO.:	1	2	3	4	5	6	7	8	9
IN./DAY:	.01	.01	.07	.41	.03	.10	.01	.03	.01

SPECIMEN TYPE: PRECUT, Undisturbed
NORMAL STRESS, T/SF: 6.0
REMARKS:

PROJECT: Smithville
BORING NO.: C-526 SAMPLE NO.: 2
DEPTH: 43.3
MRD LAB NO.: 84/205 DATE: 88 JUN 884
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 6

MRD Form 780, 1 May 80 (Replaces 1 Nov 68 edition)

Revisions			
Symbol	Descriptions	Date	Approved

U S ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by J.B.M.
Drawn by J.W.P.
Checked by J.B.M.
Submitted by P.C.M.

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT
TEST DATA
LEFT ABUTMENT, RESIDUAL S TESTS
BORING C-526 (P-110-6)

Scale AS SHOWN
MARCH 1987
File No. RP-3 1772

3 | 2

SUPPLEMENT NO. 1 PLATE NO. 22

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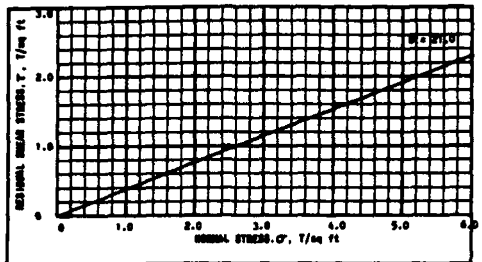
D



						RESIDUAL SHEAR STRESS, T/sq ft	2.323		
TEST NO.									
INITIAL	WATER CONTENT %	w_o	-			τ/σ AT RESIDUAL	0.387		
	VOID RATIO	e_o	-			TIME TO RESIDUAL, days	8		
	SATURATION %	S_o	-			DISPLACEMENT TO RESIDUAL, inches	2.980		
	DRY DENSITY, T/cu ft	γ_d	-			FINAL SHEAR STRESS, T/sq ft	2.71		
SPECIFIC GRAVITY		G_s	-			FINAL DISPLACEMENT, inches	3.135		
LIQUID LIMIT		LL	31*			LENGTH OF TEST, days	8		
PLASTIC LIMIT		PL	15*			MATERIAL EXTRUDED, gm	-		
WATER CONTENT %		w_f	-			MATERIAL EXTRUDED, %	-		
NORMAL STRESS, T/sq ft		σ	6.0			SHEAR STRENGTH PARAMETERS: φ MAXIMUM	32.6		
MAXIMUM SHEAR STRESS, T/sq ft		τ_{max}	3.837			φ RESIDUAL	21.2		
DISPLACEMENT TO MAXIMUM, inches			0.114			COHESION T/sq ft	-		

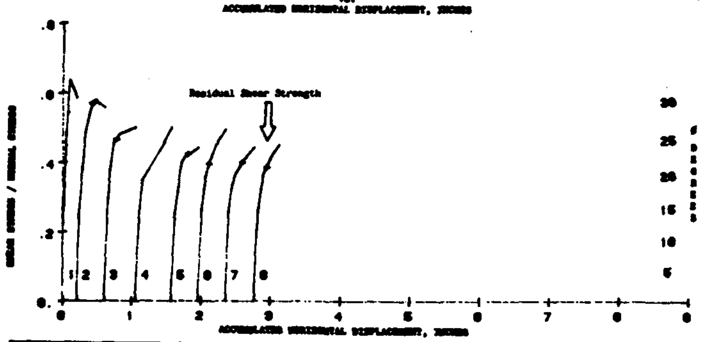
SPECIMEN DIMENSIONS 3.0 IN. SQUARE - IN. THICK	MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
CONTROLLED ☒ STRAIN ☐ STRESS	PROJECT Smithville LAB. 0304/74
TYPE OF SPECIMEN Undisturbed, Intact	AREA
CLASSIFICATION *From very small	BORING NO. 525 SAMPLE NO. 2
soil sample from residual surface.	DEPTH ELEVATION 42.6-42.8 DATE 17 MAY 1984
Pull 4 has partial data.	RESIDUAL DIRECT SHEAR TEST REPORT

ENG FORM 1 NOV 68 750

Figure 7

C

SHEAR STRESS/NORMAL STRESS
VS.
ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



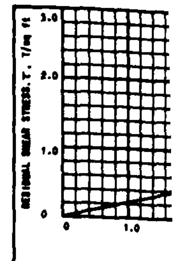
PULL NO:	1	2	3	4	5	6	7	8
IN./DAY:	.3	.40	.43	.44	.47	.43	.39	.38

SPECIMEN TYPE: INTACT
NORMAL STRESS, T/SF: 6.0
REMARKS: See Plate - 3

PROJECT: Smithville
BORING NO: US 525 SAMPLE NO: 2
DEPTH: -
MRD LAB NO: 04/74 DATE: 17 MAY 1984
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 8

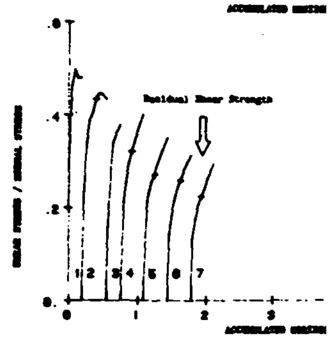
MRD Form 760, 1 May 80 (Replaces 1 Nov 68 edition)

B



TEST NO.
INITIAL
WATER CONTENT % -
VOID RATIO -
SATURATION % -
DRY DENSITY, T/cu ft -
SPECIFIC GRAVITY -
LIQUID LIMIT LL 39*
PLASTIC LIMIT PL 19*
WATER CONTENT % -
NORMAL STRESS, T/sq ft σ 6.0
MAXIMUM SHEAR STRESS, T/sq ft τ_{max} 2.59
DISPLACEMENT TO MAXIMUM, inches 0.10
SPECIMEN DIMENSIONS 3.0 IN. SQUARE - IN. THICK
CONTROLLED ☒ STRAIN ☐ STRESS
TYPE OF SPECIMEN Undisturbed, Intact
CLASSIFICATION Pull 3 has only
partial data. *From residual
surface
**From soft seam trimmings.
ENG FORM 1 NOV 68 750

SHEAR STRESS
ACCUMULATED HORIZONTAL



PULL NO:	1	2	3	4	5	6	7
IN./DAY:	.38	.33	.19	.35	.38	.37	.38

SPECIMEN TYPE: INTACT
NORMAL STRESS, T/SF: 6.0
REMARKS:

MRD Form 760, 1 May 80 (Replaces 1 Nov 68 edition)

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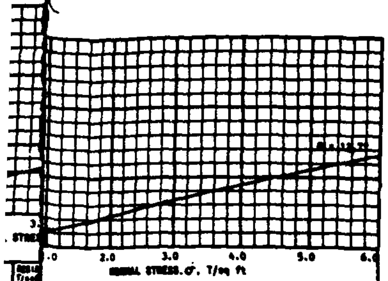
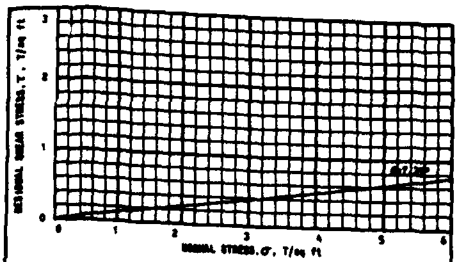
MS-1.7





Residual shear stress, T/sq ft	1.385
τ/σ at residual	0.236
Time to residual, days	7
Displacement to residual, inches	2.046
Final shear stress, T/sq ft	1.773
Final displacement, inches	2.385
Length of test, days	7

Figure 9

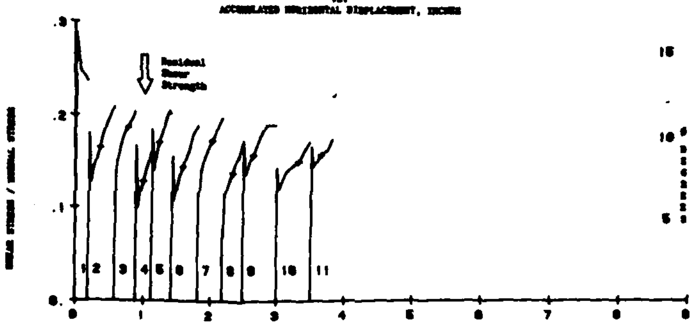
TEST NO.		RESIDUAL SHEAR STRESS, T/sq ft	.77
INITIAL WATER CONTENT	%	τ/σ AT RESIDUAL	.13
VOID RATIO	%	TIME TO RESIDUAL, days	4
SATURATION	%	DISPLACEMENT TO RESIDUAL, inches	1.00
DRY DENSITY, lb/cu ft	γ_d	FINAL SHEAR STRESS, T/sq ft	1.05
SPECIFIC GRAVITY		FINAL DISPLACEMENT, inches	3.84
LIQUID LIMIT	LL	LENGTH OF TEST, days	11
PLASTIC LIMIT	PL		
NORMAL STRESS, T/sq ft	σ 6.0	SHEAR STRENGTH PARAMETERS ϕ RESIDUAL	7.35
MAXIMUM SHEAR STRESS, T/sq ft	τ_{max} 1.73		
DISPLACEMENT TO MAXIMUM, inches	.03		

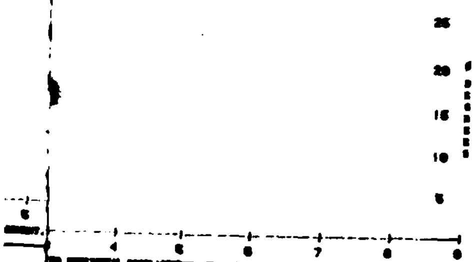
SPECIMEN DIMENSIONS 3.01 IN. SQUARE	MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
CONTROLLED ■ STRAIN ☐ STRESS	PROJECT Smithville LAB. NO. 84/205
TYPE OF SPECIMEN Undisturbed, Intact	AREA
CLASSIFICATION	BORING NO. C-520 SAMPLE NO. 1
*Atterbergs were conducted on very small quantity of some material which may effect the results.	DATE 31 JUL 1984
	RESIDUAL DIRECT SHEAR TEST REPORT

MRD Form 1 NOV 80 760

Figure 8

SHEAR STRESS/NORMAL STRESS
VS.
ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES





ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES

PULL NO:	1	2	3	4	5	6	7	8	9	10	11
IN./DAY:	.25	.3	.25	.12	.24	.4	.20	.4	.21	.25	.35

SPECIMEN TYPE: Undisturbed, Intact
NORMAL STRESS, T/SF: 6.0
REMARKS: See Plate 4

PROJECT: Smithville
SAMPLE NO: 1
DATE: 31 JUL 1984
MRD LAB NO: 84/205
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 9

MRD Form 760, 1 May 80 (Replaces 1 Nov 60 edition)

PROJECT: Smithville
SAMPLE NO: 8
MRD LAB NO: 84/74 DATE: 17 MAY 1984
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 10

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by J.B.H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT TEST DATA LEFT ABUTMENT, RESIDUAL S TEST BORINGS UC-525 (P-109-1), C-520 (P-109-2)
Drawn by J.B.H.	
Checked by J.B.H.	Scale AS SHOWN
Submitted by F.C.H.	MARCH 1987
	File No. RP-3-1773

SUPPLEMENT NO 1 PLATE NO. 25

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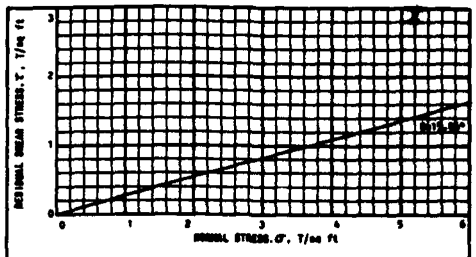
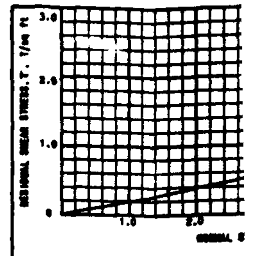


TEST NO.					RESIDUAL SHEAR STRESS, T/sq ft	1.62		
INITIAL WATER CONTENT	%				τ/σ AT RESIDUAL	.27		
VOID RATIO					TIME TO RESIDUAL, days	9		
SATURATION	%				DISPLACEMENT TO RESIDUAL, inches	3.15		
DRY DENSITY, T/cu ft	γ_d				FINAL SHEAR STRESS, T/sq ft	1.17		
SPECIFIC GRAVITY					FINAL DISPLACEMENT, inches	3.99		
LIQUID LIMIT	LL				LENGTH OF TEST, days	10		
PLASTIC LIMIT	PL				MATERIAL EXTRUDED, gms	-		
WATER CONTENT					MATERIAL EXTRUDED, %	-		
NORMAL STRESS, T/sq ft	σ	6.0			SHEAR STRENGTH PARAMETERS φ MAXIMUM	-		
MAXIMUM SHEAR STRESS, T/sq ft	τ_{max}	3.36			φ RESIDUAL	15.06		
DISPLACEMENT TO MAXIMUM, inches		.12			COHESION T/sq ft	-		

SPECIMEN DIMENSIONS 3.00 IN. SQUARE IN. THICK	MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
CONTROLLED ☒ STRAIN ☐ STRESS	PROJECT Smithville LAB. NO. 84/205
TYPE OF SPECIMEN Undisturbed, Intact	AREA
CLASSIFICATION There was not enough	BORING NO. UC 525 / SAMPLE NO. 1, soft seam
material to determine the	DEPTH ELEVATION 40.1 / DATE [illegible]
Atterberg Limits. See Plate 1.	RESIDUAL DIRECT SHEAR TEST REPORT

MRD FORM 1 NOV 80 760

Figure 1



TEST NO.			
INITIAL WATER CONTENT	%		
VOID RATIO		-	
SATURATION	%	-	
DRY DENSITY, T/cu ft	γ_d	-	
SPECIFIC GRAVITY		-	
LIQUID LIMIT	LL	52*	
PLASTIC LIMIT	PL	18*	
WATER CONTENT			
NORMAL STRESS, T/sq ft	σ	6.0	
MAXIMUM SHEAR STRESS, T/sq ft	τ_{max}	2.40	
DISPLACEMENT TO MAXIMUM, inches		0.10	

SPECIMEN DIMENSIONS 3.0 IN. SQUARE IN. THICK

CONTROLLED ☒ STRAIN ☐ STRESS

TYPE OF SPECIMEN Undisturbed, Intact

CLASSIFICATION Specimen was sheared on existing concave shaped seam. Considerable material was extruded during the test. *From very small amount of soil from residual surface.

MRD FORM 1 NOV 80 760

C

B

SHEAR STRESS/NORMAL STRESS VS. ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES

PULL NO:	1	2	3	4	5	6	7	8	9	10
IN./DAY:	[illegible]	[illegible]	.19	[illegible]	.44	.44	[illegible]	.11	[illegible]	.4

SPECIMEN TYPE: Undisturbed, Intact

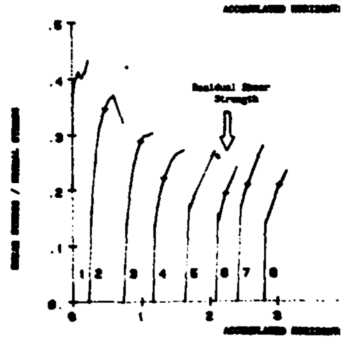
NORMAL STRESS, T/SF: 6.0

REMARKS:

PROJECT: Smithville
BORING NO: [illegible] SAMPLE NO: 1, Soft seam
DEPTH: 40.1
MRD LAB NO: 84/205 DATE: [illegible]
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 2

MRD Form 760, 1 May 80 (Replaces 1 Nov 80 edition)

SHEAR STRESS/NORMAL STRESS VS. ACCUMULATED HORIZONTAL DISPLACEMENT



PULL NO:	1	2	3	4	5	6	7	8
IN./DAY:	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SPECIMEN TYPE: [illegible]

NORMAL STRESS, T/SF: 6.0

REMARKS: See Plate 1; *Partial data only.

MRD Form 760, 1 May 80 (Replaces 1 Nov 80 edition)

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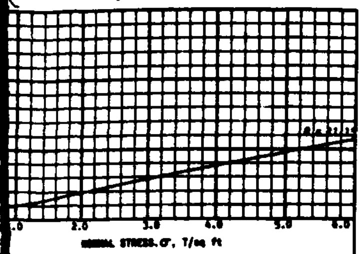
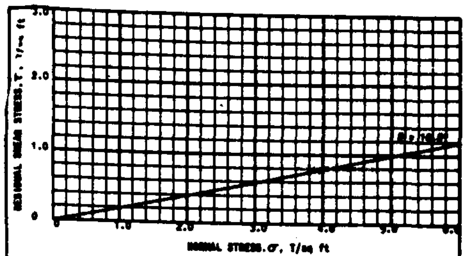


[illegible]				RESIDUAL SHEAR STRESS, T/sq ft	1.170			
[illegible]				[illegible] AT RESIDUAL	0.196			
[illegible]	-			TIME TO RESIDUAL, days	6			
[illegible]	-			DISPLACEMENT TO RESIDUAL, inches	2.250			
[illegible]	-			FINAL SHEAR STRESS, T/sq ft	1.479			
[illegible]	-			FINAL DISPLACEMENT, inches	3.198			
[illegible]	52°			LENGTH OF TEST, days	8			
[illegible]	18°			MATERIAL EXTRUDED, gm	-			
[illegible]				MATERIAL EXTRUDED, %	-			
[illegible]	6.0			SHEAR STRENGTH PARAMETERS	φ [illegible]	22.6		
[illegible]	2.494				φ RESIDUAL	11.1		
[illegible]	0.104				COHESION T/sq ft	-		

IN. THICK	MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
☐ STRESS	PROJECT Smithville — LAB. NO. 84/74
[illegible] turbed, Intact	AREA
[illegible] was sheared [illegible]	BORING NO. 525 — SAMPLE NO. 1, Slick
[illegible]	DEPTH ELEVATION 41.0-41.5 — DATE 17 MAY 1984
[illegible]	RESIDUAL DIRECT SHEAR TEST REPORT

Figure 3



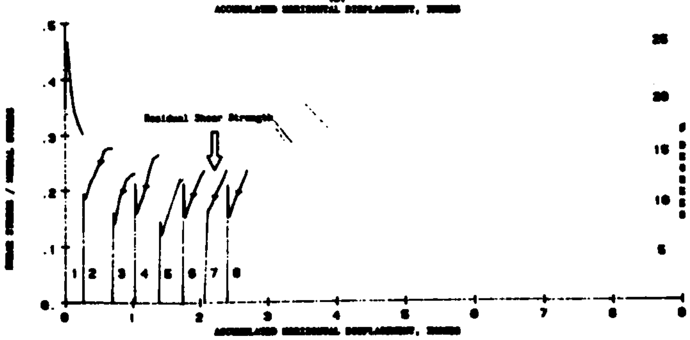
TEST NO.					RESIDUAL SHEAR STRESS, T/sq ft	1.193		
INITIAL	WATER CONTENT %	w_o			[illegible] AT RESIDUAL	0.191		
	VOID RATIO	e_o	-		TIME TO RESIDUAL, days	7		
	SATURATION %	S_o	-		DISPLACEMENT TO RESIDUAL, inches	2.238		
	DRY DENSITY, T/cu ft	γ_d	-		FINAL SHEAR STRESS, T/sq ft	1.419		
SPECIFIC GRAVITY		G_s	-		FINAL DISPLACEMENT, inches	2.710		
LIQUID LIMIT		LL	51°		LENGTH OF TEST, days	8		
PLASTIC LIMIT		PL	17°		MATERIAL EXTRUDED, gm	-		
WATER CONTENT %		w_f	-		MATERIAL EXTRUDED, %	-		
NORMAL STRESS, T/sq ft		σ	6.0		SHEAR STRENGTH PARAMETERS φ [illegible]	25.1		
MAXIMUM SHEAR STRESS, T/sq ft		τ_{max}	2.812		φ RESIDUAL	10.8		
DISPLACEMENT TO MAXIMUM, inches			0.046		COHESION T/sq ft	-		

SPECIMEN DIMENSIONS 3.0 IN. SQUARE IN. THICK	MISSOURI RIVER DIVISION LABORATORY, OMAHA, NEBRASKA
CONTROLLED ☒ STRAIN ☐ STRESS	PROJECT Smithville — LAB. NO. 84/74
TYPE OF SPECIMEN Undisturbed, Intact	AREA
CLASSIFICATION From very small soil sample from residual surface. Pull 5 has partial data.	BORING NO. 525 — SAMPLE NO. 1, No slick
	DEPTH ELEVATION 41.0-41.5 — DATE 17 MAY 1984
	RESIDUAL DIRECT SHEAR TEST REPORT

MRD FORM 1 NOV 68 700

Figure 5

SHEAR STRESS/NORMAL STRESS
VS.
ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



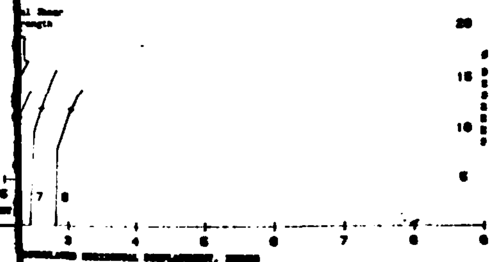
PULL NO:	1	2	3	4	5	6	7	8
IN./DAY:	.20	.40	.20	.24	.20	.20	.24	.20

SPECIMEN TYPE: [illegible]
NORMAL STRESS, T/SF: 6.0
REMARKS: See Plate - 2

PROJECT: Smithville
BORING NO: [illegible] SAMPLE NO: 1, No slick
DEPTH:
MRD LAB NO: 84/74 DATE: 17 MAY 1984
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 6

MRD Form 700, 1 May 80 (Replaces 1 Nov 68 edition)

SHEAR STRESS/NORMAL STRESS
VS.
ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



[illegible]	5	6	7	8
[illegible]	.20	.40	.40	.20

PROJECT: Smithville
BORING NO: [illegible] SAMPLE NO: 1, Slick
DEPTH:
MRD LAB NO: 84/74 DATE: 17 MAY 1984
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 4

[illegible] 80 edition)

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by J.B.H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by J.W.R	TEST DATA LEFT ABUTMENT, RESIDUAL S TESTS BORING UC-525 (P-109-I)
Checked by J.B.H	Scale AS SHOWN
Submitted by F.C.W.	MARCH 1987 — File No. RP-3-1774

SUPPLEMENT NO. 1 PLATE NO. 24

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2

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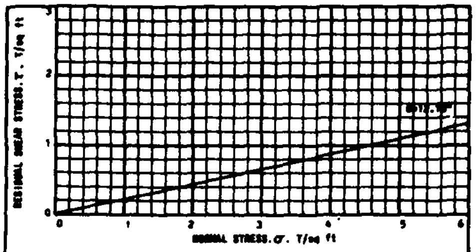
D

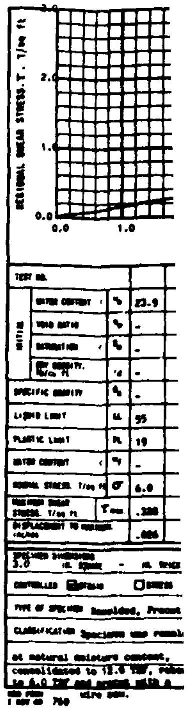


TEST NO.						RESIDUAL SHEAR STRESS, T/sq ft	1.29		
INITIAL	WATER CONTENT	%				T/σ AT RESIDUAL	.22		
	VOID RATIO	%				TIME TO RESIDUAL, days	10		
	SATURATION	%				DISPLACEMENT TO RESIDUAL, inches	3.98		
	DRY DENSITY, lb/cu ft	γ_d				FINAL SHEAR STRESS, T/sq ft	1.42		
SPECIFIC GRAVITY		%				FINAL DISPLACEMENT, inches	3.68		
LIQUID LIMIT		LL	49			LENGTH OF TEST, days	10		
PLASTIC LIMIT		PL	19			MATERIAL EXTRUDED, gm			
WATER CONTENT		w_f				MATERIAL EXTRUDED,			
NORMAL STRESS, T/sq ft		σ	6.0			SHEAR STRENGTH PARAMETERS — φ PEAK			
MAXIMUM SHEAR STRESS, T/sq ft		T_{max}	3.36			SHEAR STRENGTH PARAMETERS — φ RESIDUAL	12.13		
DISPLACEMENT TO MAXIMUM, inches			.08			COHESION T/sq ft			

SPECIMEN DIMENSIONS 3.01 IN. SQUARE IN. THICK	MISSOURI RIVER DIVISION LABORATORY OMAHA NEBRASKA
CONTROLLED ☒ STRAIN ☐ STRESS	PROJECT Smithville LAB. NO. 84/205
TYPE OF SPECIMEN Undisturbed, Intact	AREA
CLASSIFICATION Atterberg Limits were	BORING NO. C-529 SAMPLE NO. 2
determined from material taken	DEPTH ELEVATION 56.0' DATE 23 JUL 1984
below the contact zone: about 20 grams material.	RESIDUAL DIRECT SHEAR TEST REPORT

MRD FORM 1 NOV 80 769

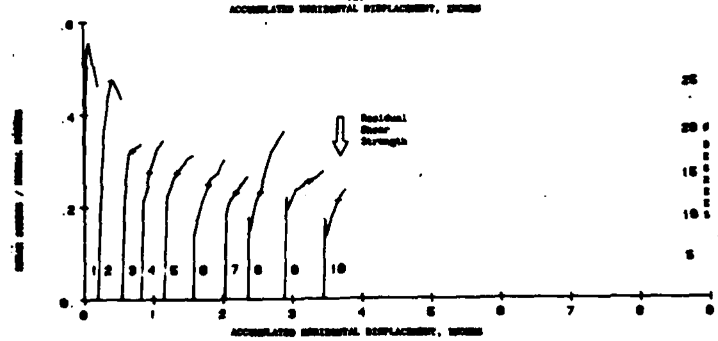
Figure 10



C

B

SHEAR STRESS/NORMAL STRESS VS. ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



PULL NO:	1	2	3	4	5	6	7	8	9	10
IN./DAY:	.20	.27	.14	.27	.42	.45	.42	.34	.35	.41

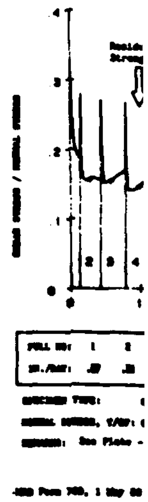
SPECIMEN TYPE: UNDISTURBED, INTACT

NORMAL STRESS, T/SF: 6.0

REMARKS: See Plate 4.

PROJECT: Smithville
BORING NO: C529 SAMPLE NO: 2
DEPTH: 56.0
MRD LAB NO: 84/205 DATE: 23 JUL 1984
RESIDUAL DIRECT SHEAR TEST REPORT
FIGURE: 11

MRD Form 769, 1 May 80 (Replaces 1 Nov 68 edition)



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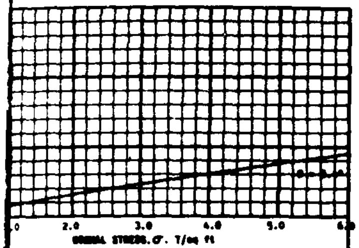
3 | 2 | 1



23.9 | % AT RESIDUAL | 0.147

FINAL SHEAR STRESS, T/sq ft | 0.905

DISPLACEMENT TO RESIDUAL, inches | 1.630

MISSOURI RIVER DIVISION LABORATORY OMAHA, NEBRASKA

PROJECT Smithville | LAB NO. 84/74

SAMPLE NO. I-109-1 | DATE 17 MAY 1984

RESIDUAL DIRECT SHEAR TEST REPORT

DIRECT SHEAR & RESIDUAL SHEAR RESULTS=SUMMARY

BORING ID	SAMPLE NO.	DEPTH	GEOLOGICAL MEMBER	NORMAL STRESS (tsf)	DRAINED SHEAR STRENGTH PEAK tan θ	DRAINED SHEAR STRENGTH RESIDUAL tan θR
I-109-1	-	38.7-42.0	RAYTOWN-THICK SHALE SEAM	6.0	.329*	.149*
P-109-1(C-525)	1	40.1	RAYTOWN-SHALE SEAM	6.0	.560	.269
P-109-1(C-525)	1	41.0-41.5	RAYTOWN-THICK SHALE SEAM W/SLICKENSIDES	6.0	.46	.186
P-109-1(C-525)	1	41.0-41.5	RAYTOWN-THICK SHALE SEAM W/SLICKS	6.0	.468	.181
P-109-1(C-525)	2	42.6-42.8	RAYTOWN/MUNCIE CREEK SHALE	6.0	.640	.300***
P-109-1(C-525)	8	51.4-51.6	CHANUTE SHALE	6.0	.499	.225***
P-109-2(C-529)	2	56.0	MUNCIE CREEK SHALE	6.0	.509	.25***
P-109-4(C-527)	2	41.3	RAYTOWN-THIN SHALE SEAM	6.0	.500	.226***
P-109-4(C-527)	3	43.8	RAYTOWN/MUNCIE CREEK SHALE	6.0	.65	.267***
P-109-3(C-530)	1	42.0	RAYTOWN-THICK SHALE SEAM	6.0	.288	.189***
P-110-9(C-536)	2	43.3	RAYTOWN-THICK SHALE SEAM	6.0	.230**	.130**
P-110-9(C-536)	2	43.3	RAYTOWN-THICK SHALE SEAM	6.0	.297**	.13**
P-110-9(C-536)	4	47.2	MUNCIE CREEK SHALE	6.0	.423	.202**
P-110-10(C-532)	1	54.0	RAYTOWN-THICK SHALE SEAM	4.0	.313	X

* REMOLDED SPECIMEN

** PRECUT SPECIMEN

*** DID NOT DEVELOP RESIDUAL CONDITION

D

C

B

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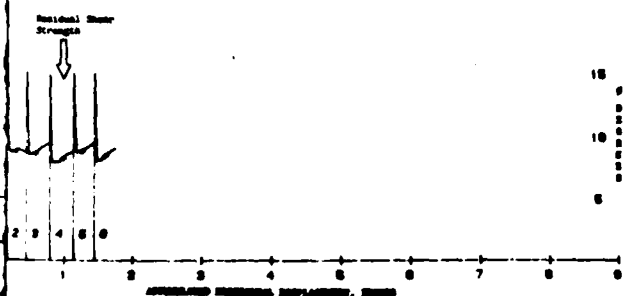


PROJECT: Smithville | I-109-1

LAB NO: 84/74 | DATE: 17 MAY 1984

FIGURE: 2

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT
TEST DATA
LEFT ABUTMENT RESIDUAL S TESTS
BORINGS C-529 (P-109-2), I-109-1
SUMMARY OF TESTS

Designed by J.B.H.

Drawn by J.W.R.

Checked by J.B.H.

Submitted by F.C.W.

Scale AS SHOWN

MARCH 1987

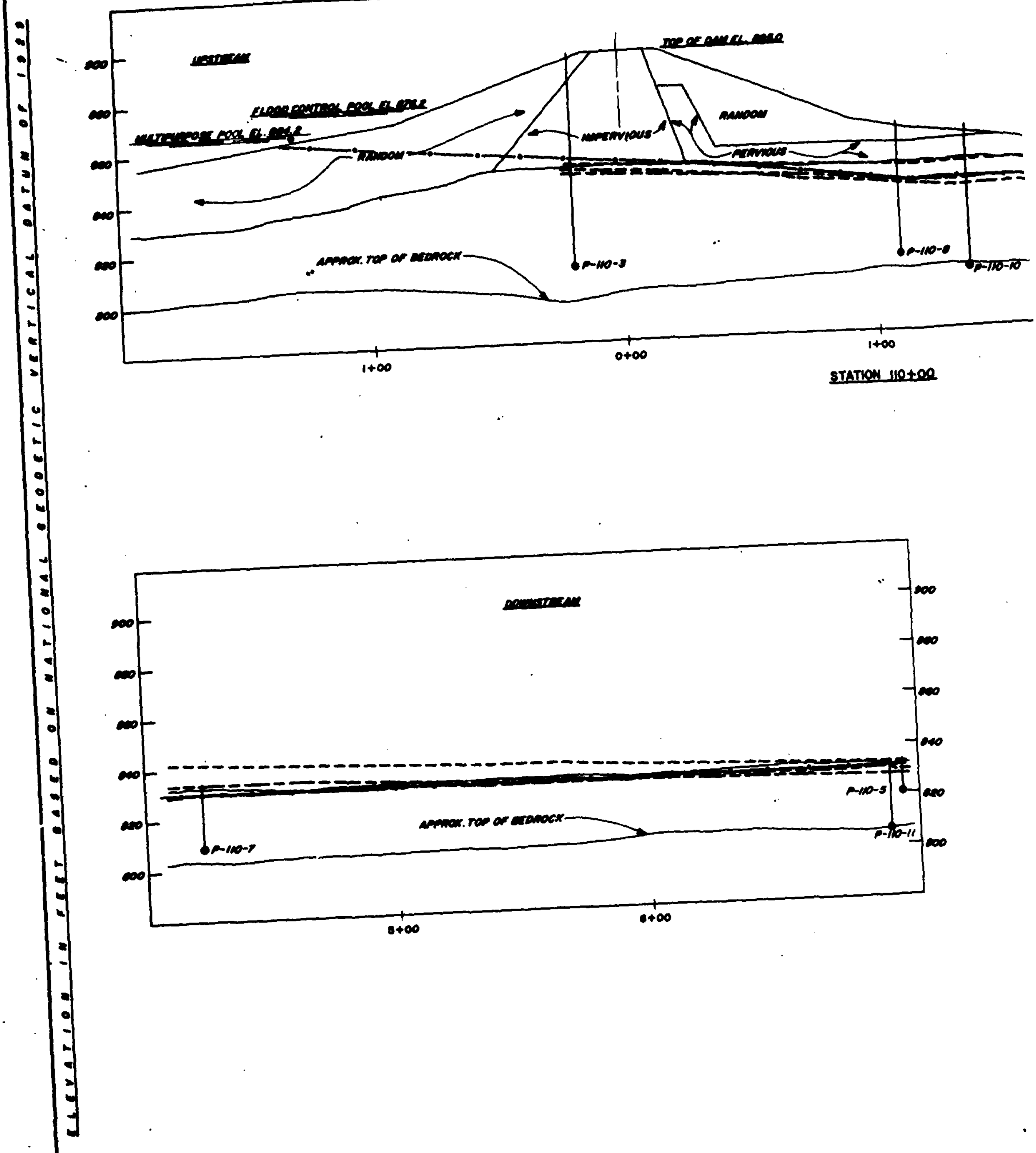
File No. RP-3-1775

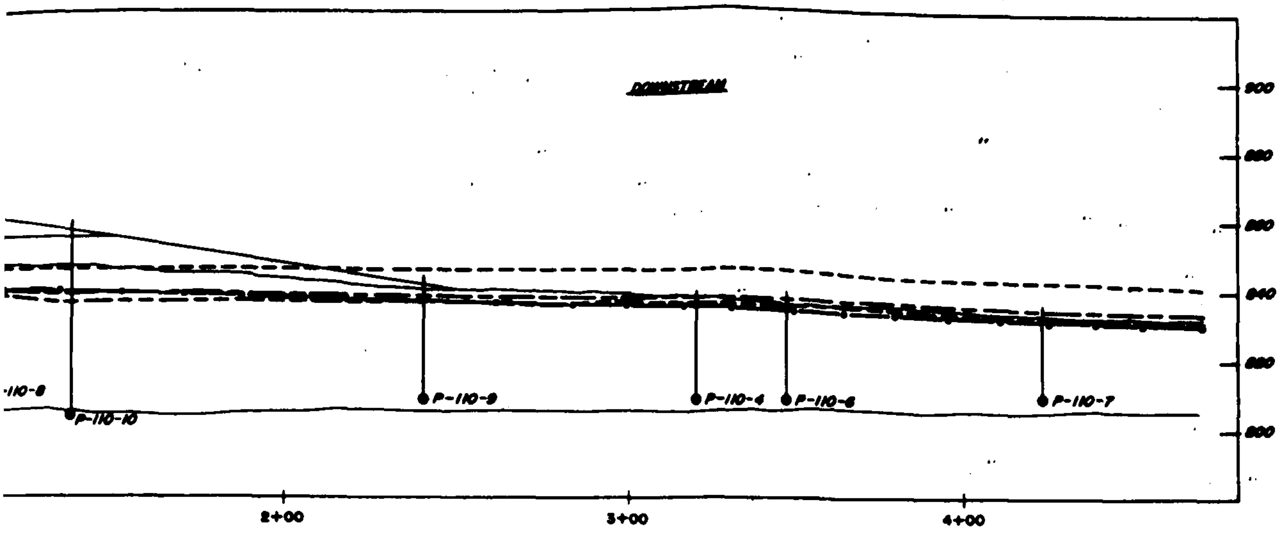
3 | 2

SUPPLEMENT NO. 1 | PLATE NO. 25

1

2


ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929
UPSTREAM
TOP OF DAM EL.
FLOOD CONTROL POOL EL.
MULTIPURPOSE POOL EL.
RANDOM
IMPERVIOUS
RANDOM
PERVIOUS
APPROX. TOP OF BEDROCK
P-110-3
P-110-8
P-110-10
900
880
860
840
820
800
1+00
0+00
1+00
STATION 110+00
DOWNSTREAM
APPROX. TOP OF BEDROCK
P-110-7
P-110-5
P-110-11
900
880
860
840
820
800
5+00
6+00



NOTES:

- 13 FEB. 1984 – POOL EL. 864.0 (BEFORE WELLS INSTALLED)
- 3 JUN 1986 – POOL EL. 866.0 (AFTER WELLS INSTALLED)
- 17 OCT. 1986 – POOL EL. 873.17 (RECORD POOL)
- PROJECTED SPILLWAY CREST POOL EL. 880.2

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

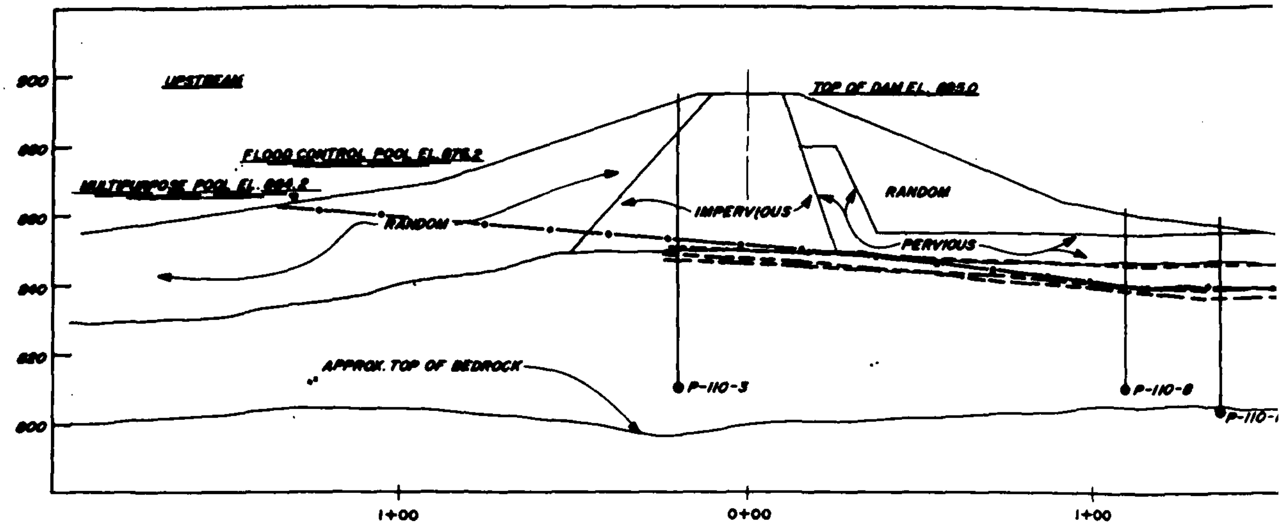
Designed by: J. D. H.
Drawn by: J.W.P.
Checked by: J.D.H.
Submitted by: F. C. W.

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT
PIEZOMETRIC LEVEL AT STA. 110+00 BEFORE & AFTER WELLS INSTALLED AT RECORD POOL, AND AT PROJECTED SPILLWAY CREST

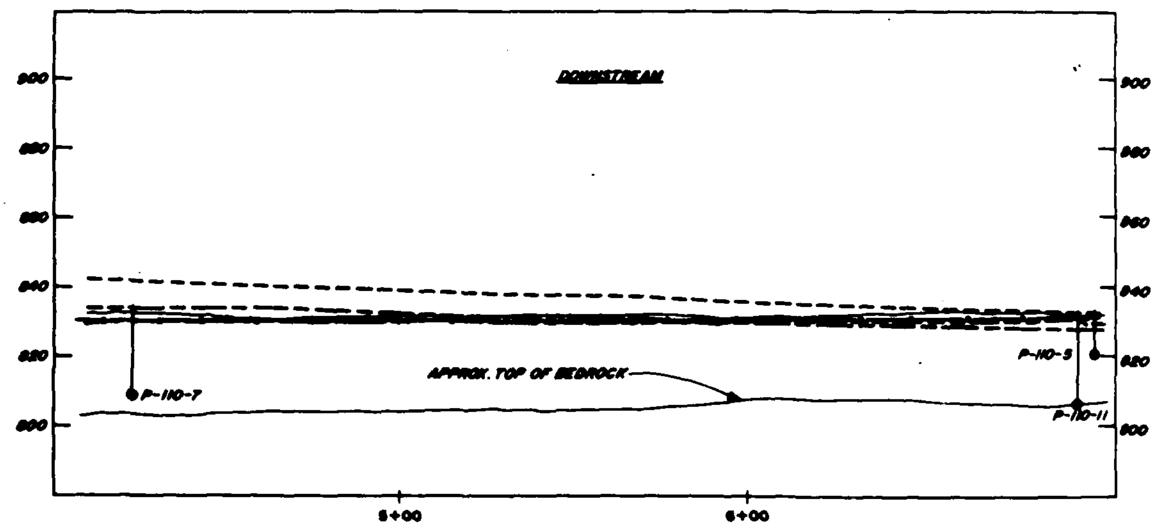
Scale: AS SHOWN
MARCH 1987
File No: RP-3-1776

SUPPLEMENT NO 1 PLATE NO. 26

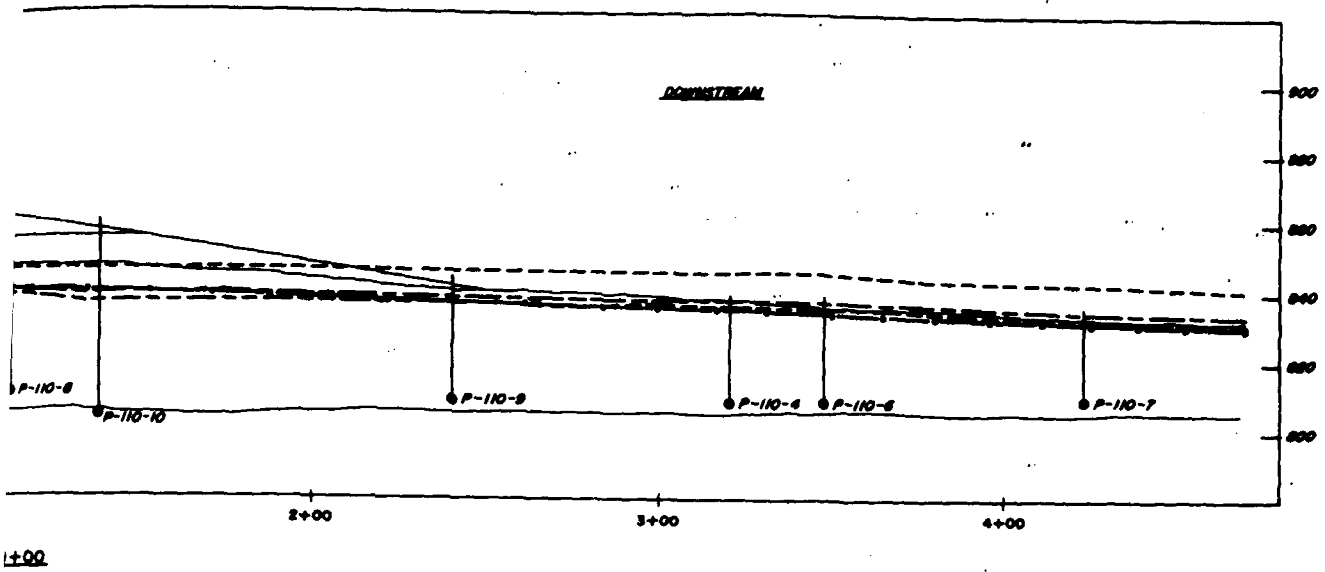
2



STATION 110+00



ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929



900
880
860
840
820
800

NOTES:

— — — — —	13 FEB. [illegible] – POOL EL. [illegible] (BEFORE WELLS INSTALLED)
——— — ———	3 JUN [illegible] – POOL EL. [illegible] (AFTER WELLS INSTALLED)
—— – — ——	17 OCT. [illegible] – POOL EL. 873.17 (RECORD POOL)
—•——•——•—	PROJECTED SPILLWAY CREST POOL EL. [illegible]

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

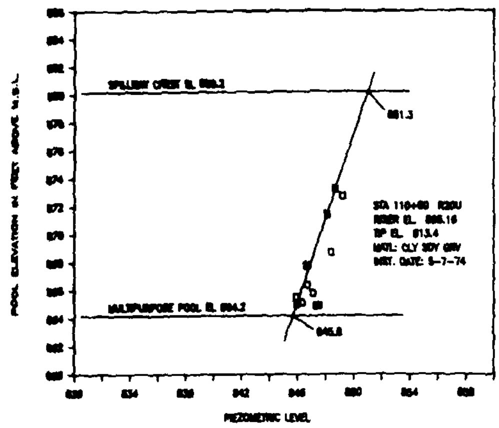
Designed by: J. G. H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by: J.W.P.	PIEZOMETRIC LEVEL AT STA. 110+00 BEFORE & AFTER WELLS INSTALLED AT RECORD POOL, AND AT PROJECTED SPILLWAY CREST
Checked by: J.D.H.	Scale: AS SHOWN
Submitted by: F.C.W.	MARCH 1987
	File No: RP-3-1776

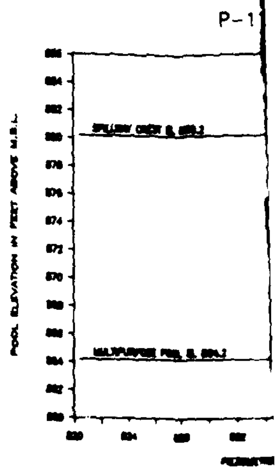
SUPPLEMENT NO 1 PLATE NO. 26

5 4

D

P-110-3





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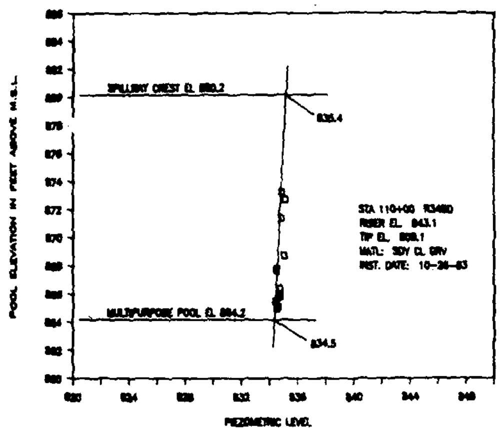
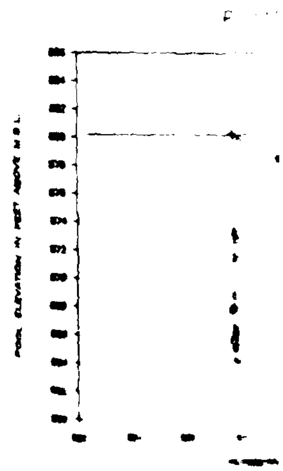
P-110-6





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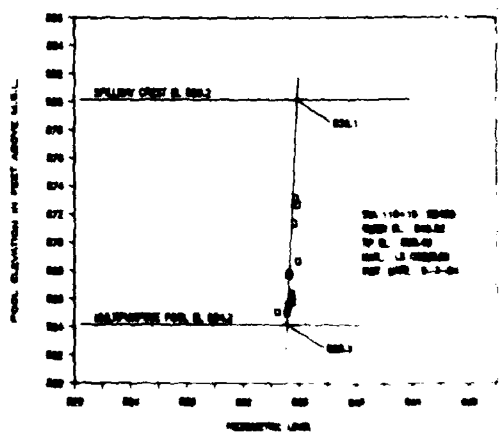
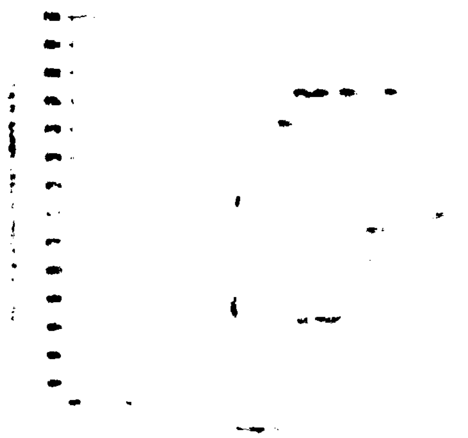
P-110-9





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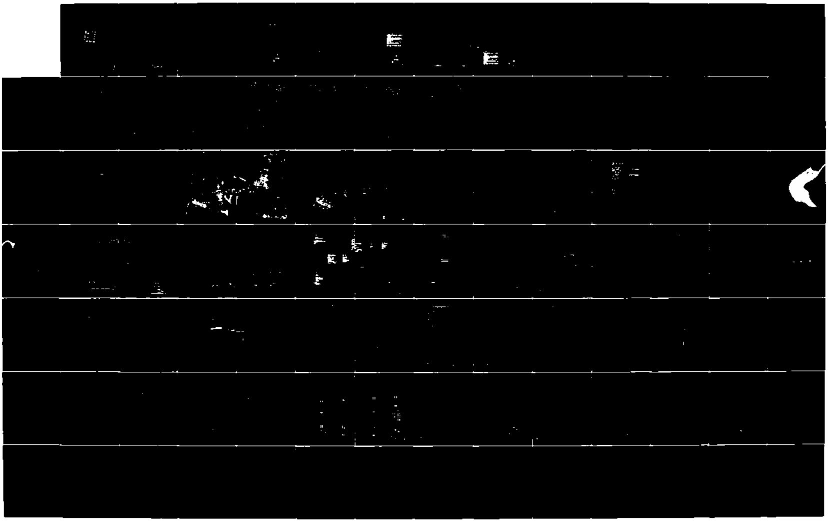
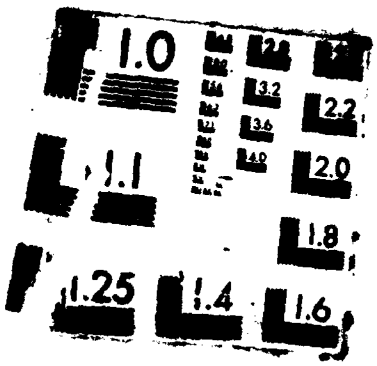
5

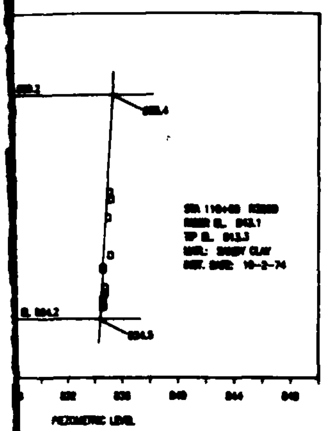
AD-A182 106 MULTIPLE-PURPOSE PROJECT: PLATTE RIVER BASIN LITTLE PLATTE RIVER MISSOURI..(U) CORPS OF ENGINEERS KANSAS CITY MO KANSAS CITY DISTRICT F C WALBERG ET AL. MAR 87 2/3

UNCLASSIFIED F/G 13/2 NL

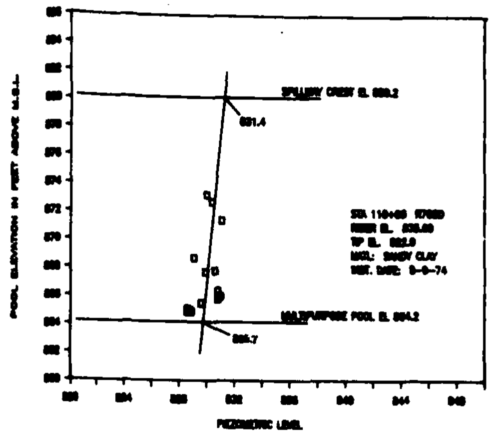



1.0
3.2
2.2
3.6
4.0
2.0
1.1
1.8
1.25
1.4
1.6

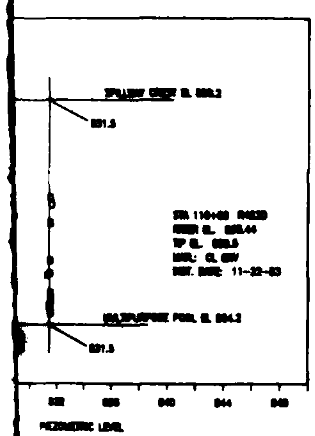
P-110-4



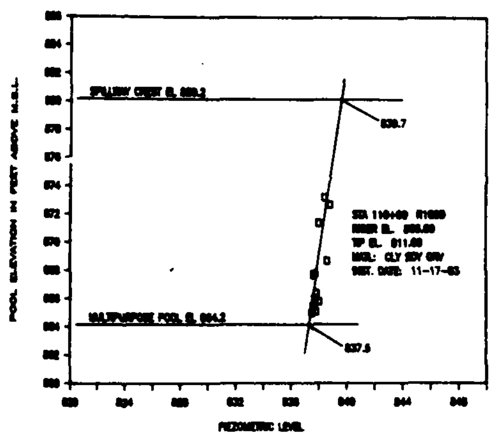
P-110-5



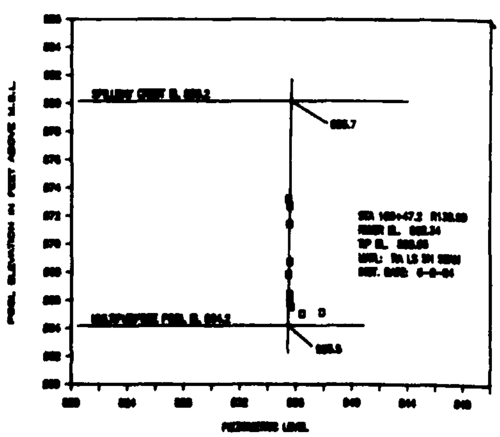
P-110-7



P-110-8



P-110-10



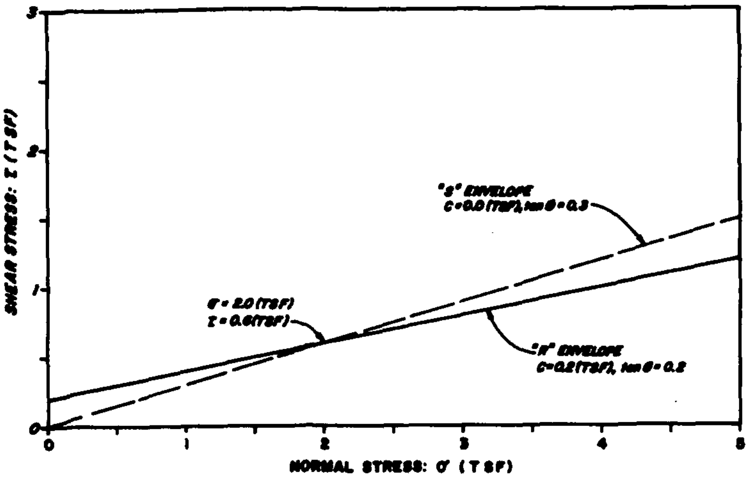
NOTE: ALL PIEZOMETER READINGS TAKEN AFTER INSTALLATION OF RELIEF WELLS.

Revisions			
Symbol	Descriptions	Date	Approved

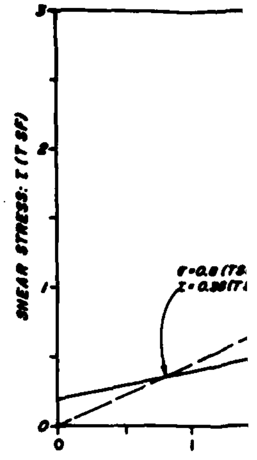
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

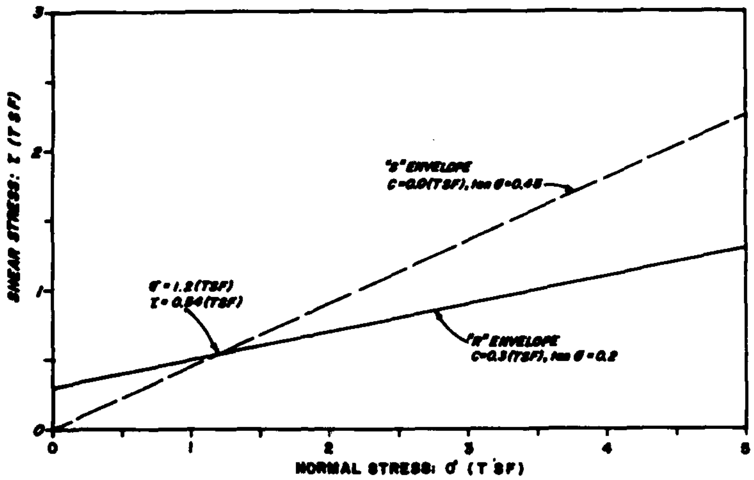
Designed by J. D. H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT PROJECTED PIEZOMETRIC LEVELS FROM PIEZOMETER DATA OBTAINED AFTER INSTALLATION OF RELIEF WELLS-STA.110+00
Drawn by J.W.P.	
Checked by J.D.H.	Scale: AS SHOWN
Submitted by F.C.W.	MARCH 1987 — File No. RP-3-1777

SUPPLEMENT NO 1 PLATE NO.27

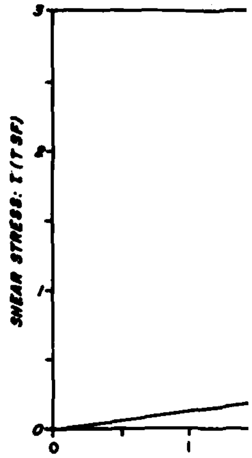


BERM FILL

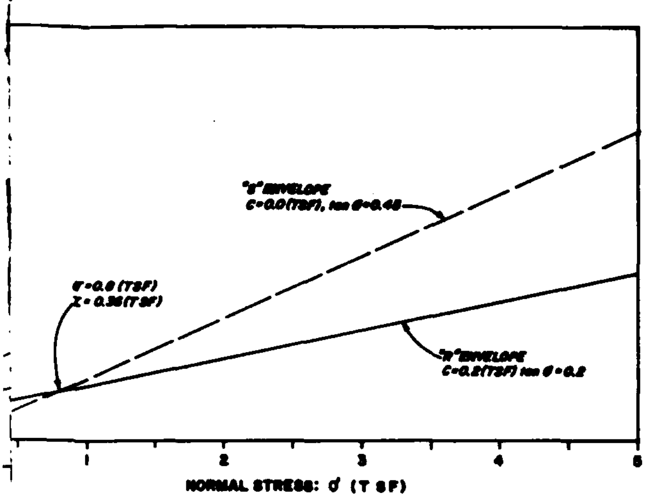




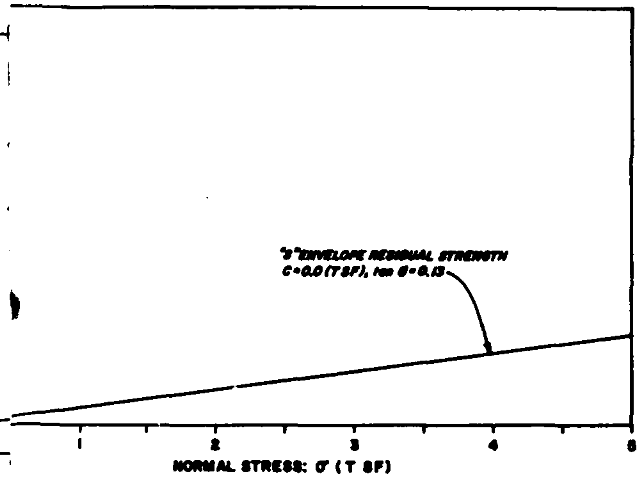
UPPER FOUNDATION OVERBURDEN



SHAL



COMPACTED FILL



SHALE SEAM IN THE RAYTOWN LIMESTONE

PHYSICAL SOIL CONSTANTS (1)						
MATERIAL	UNIT WEIGHT (LBS/FT³)		SHEAR STRENGTH PARAMETERS			
			"R"		"S"	
	MOIST	SATURATED	C (TSF)	tan Ø	C (TSF)	tan Ø
BERM FILL	110	115	0.2	0.2	0.0	0.3
COMPACTED FILL	120	125	0.2	0.2	0.0	0.46
UPPER FOUNDATION OVERBURDEN	115	120	0.3	0.2	0.0	0.46

PHYSICAL SOIL CONSTANTS (2)			
MATERIAL	UNIT WEIGHT (SATURATED) (LBS/FT³)	SHEAR STRENGTH PARAMETERS	
		C(TSF)	tan Ø
LOWER (COARSER) FOUNDATION OVERBURDEN	120	0	0.58
RAYTOWN LIMESTONE (3)	140	0	0
SHALE SEAM IN THE RAYTOWN LIMESTONE (4)	140	0	0.13

NOTES:

(1) SOIL CONSTANTS REFERENCED IN DESIGN MEMORANDUM #10, "SOIL DATA AND EMBANKMENT DESIGN", APPENDIX B PLATE NO. 2.

(2) SOIL CONSTANTS REFERENCED IN THE "LEFT ABUTMENT STABILITY REPORT", JULY 1984, (PLATE NO. A-27).

(3) ZERO SHEAR STRENGTH IS ASSIGNED FOR THE RAYTOWN LIMESTONE MEMBER. THE THIN AND VERTICALLY JOINTED MEMBER IS ASSUMED TO BE INCAPABLE OF RESISTING SHEARING FORCES.

(4) RESIDUAL STRENGTH IS LISTED FOR THE SHALE SEAM IN THE RAYTOWN LIMESTONE. (PEAK STRENGTHS ARE LISTED FOR THE REMAINDER OF MATERIALS).

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: M.S.G.
Drawn by: P.S.T.
Checked by: J.D.M.
Submitted by: F.C.W.

LITTLE PLATTE RIVER, MISSOURI - SMITHVILLE LAKE

EMBANKMENT CRITERIA REPORT

SHEAR STRENGTH PARAMETERS (LEFT ABUTMENT)

Scale: AS SHOWN

MARCH 1987

File No. RP-3-1778

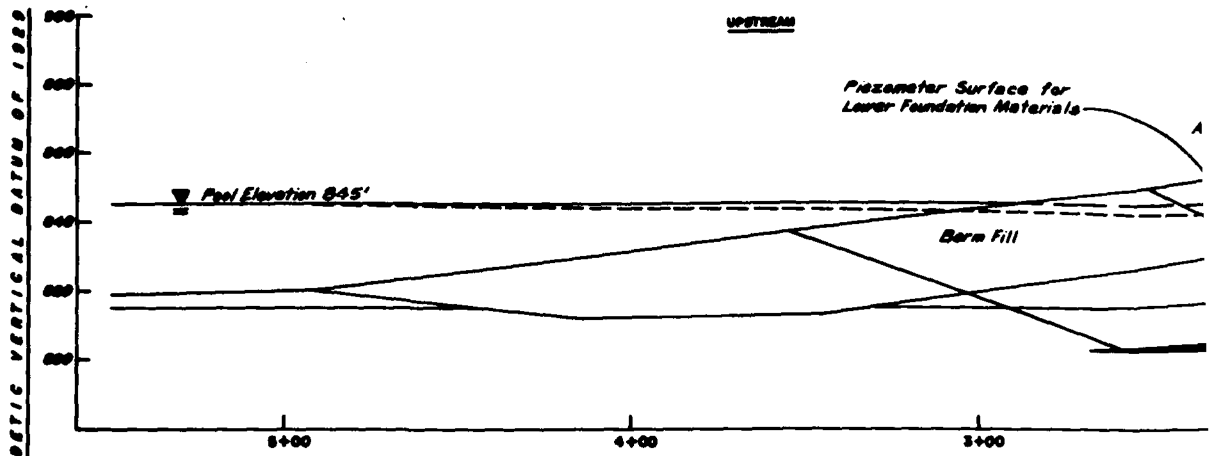
SUPPLEMENT NO 1

PLATE NO. 28

2

5	4	

D



STATION 11(

LEFT ABUTMENT SECTI(

C

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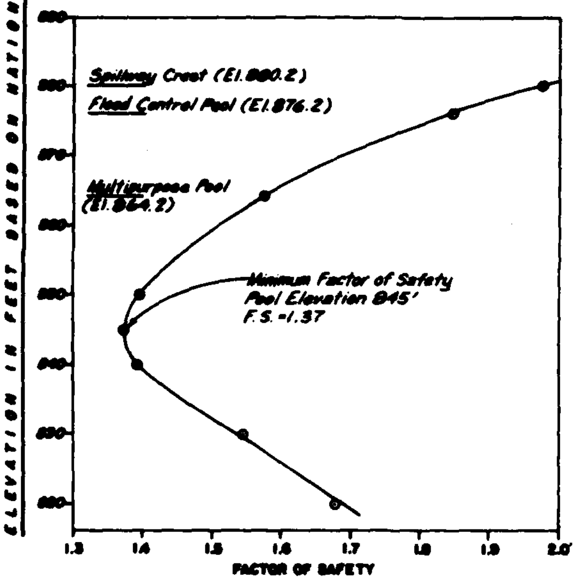
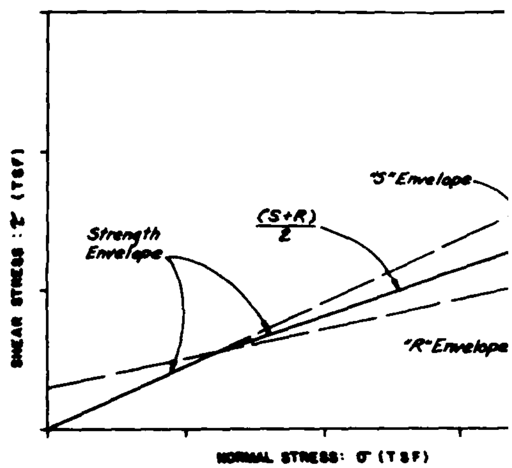


PARTIAL POOL CASE

SAFETY FACTOR VS POOL ELEVATION

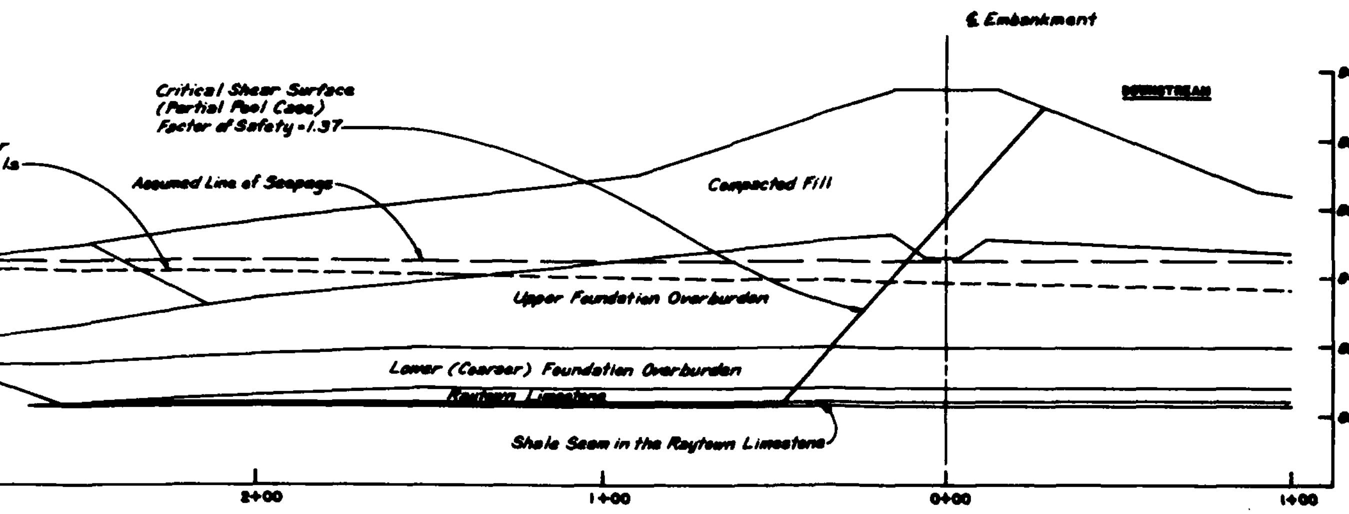


TYPICAL INTERMEDIATE ENVELOPE

(Used for Partial Pool and Steady Seepag

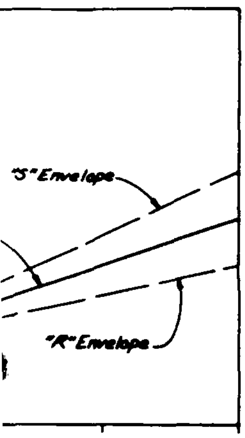
A

5	4	



STATION 110+00

...UTMENT SECTION - PARTIAL POOL CASE



...T (TSF)

...ENVELOPE

...d Steady Seepage Cases)

RESULTS OF THE PARTIAL POOL STABILITY ANALYSIS

POOL ELEVATION	FACTOR OF SAFETY
NO POOL	1.68
830	1.56
840	1.39
845	1.37
850	1.40
(MULTIPURPOSE) 864.2	1.57
(FLOOD CONTROL) 876.2	1.66
(SPILLWAY CREST) 880.2	1.57

NOTES

1. SHEAR STRENGTH PARAMETERS FOR SOILS COMPRISING THE LEFT ABUTMENT ARE REFERENCED ON PLATE NO. 26.
2. SHEAR STRENGTHS FOR MOST OF THE MATERIALS WERE DERIVED FROM THE STRENGTH ENVELOPES SHOWN ON THIS PLATE. THE S STRENGTH WAS ASSIGNED TO THE LOWER (COARSER) FOUNDATION OVERBURDEN MATERIAL AND THE RESIDUAL STRENGTH WAS ASSIGNED TO THE SHALE SEAM IN THE RAYTOWN LIMESTONE.
3. THE ASSUMED LINE OF SEEPAGE IS USED TO DETERMINE WATER PRESSURES ACTING ON THE COMPACTED FILL AND BERM FILL MATERIALS.
4. THE PIEZOMETRIC SURFACE IS USED TO DETERMINE WATER PRESSURES ACTING ON LOWER (COARSER) FOUNDATION OVERBURDEN AND BEDROCK MATERIALS. THE PIEZOMETRIC SURFACE IS DERIVED FROM WATER LEVELS WHICH WERE READ IN A LINE OF PIEZOMETERS AT STATION 110+00 AFTER THE INSTALLATION OF REMEDIAL MEASURES AND PROJECTED TO THE REQUIRED POOL ELEVATIONS.
5. WATER PRESSURES FOR THE UPPER FOUNDATION OVERBURDEN MATERIAL ARE INTERPOLATED VALUES FROM THE ASSUMED LINE OF SEEPAGE AND PIEZOMETRIC SURFACE.
6. THE RAYTOWN LIMESTONE BEDROCK MEMBER IS SHOWN TO EXTEND 267 FEET UPSTREAM. THIS UPSTREAM LIMIT WAS DETERMINED FROM THE TOP OF BEDROCK CONTOURS AND SUBCROP MAP (SEE PLATE NO. A-8) AND FROM THE DRILL LOG DETAILING THE INSTALLATION OF PIEZOMETER P-110-2 (SEE PLATE NO. 12).
7. A COMPUTER PROGRAM ENTITLED UTEXAS2 WAS USED TO PERFORM THE STABILITY ANALYSES. OPTIONS WERE SELECTED WHICH ALLOWED THE PROGRAM TO SEARCH FOR THE SHEAR SURFACE CORRESPONDING TO A MINIMUM FACTOR OF SAFETY AND TO SOLVE FOR THE FACTOR OF SAFETY UTILIZING SPENCER'S PROCEDURE.

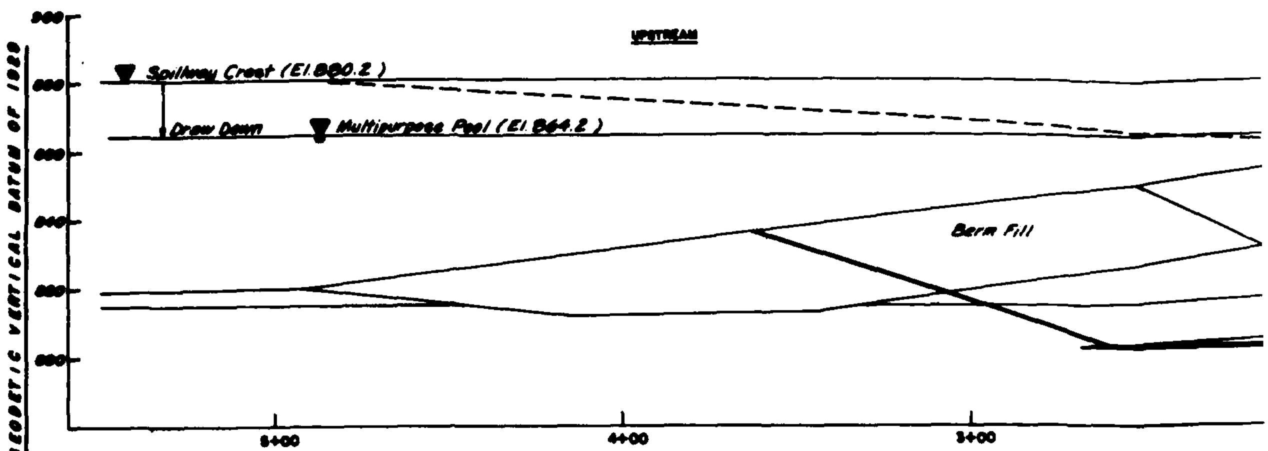
Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

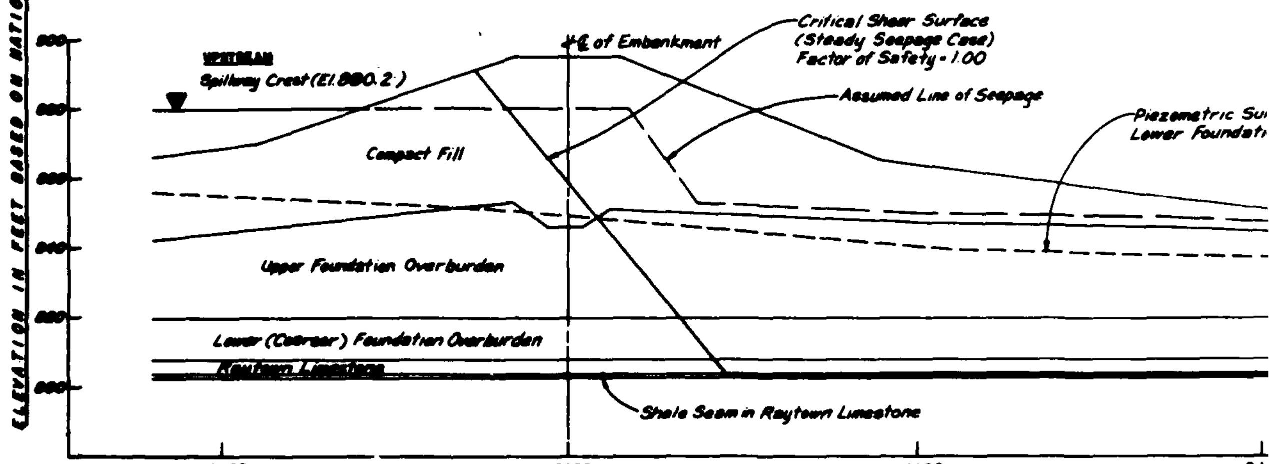
Designed by: M.S.S.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by: V.A.S.	PARTIAL POOL STABILITY ANALYSIS (LEFT ABUTMENT)
Checked by: J.B.H.	Scale: AS SHOWN
Submitted by: F.C.W.	MARCH 1987
	File No. RP-3-1779

SUPPLEMENT NO 1 PLATE NO. 29

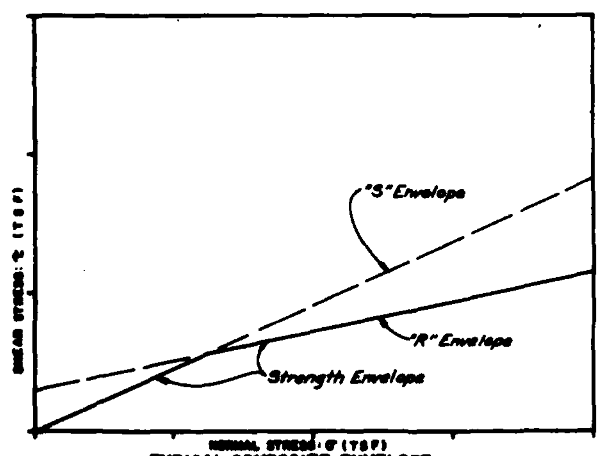
5 4 3



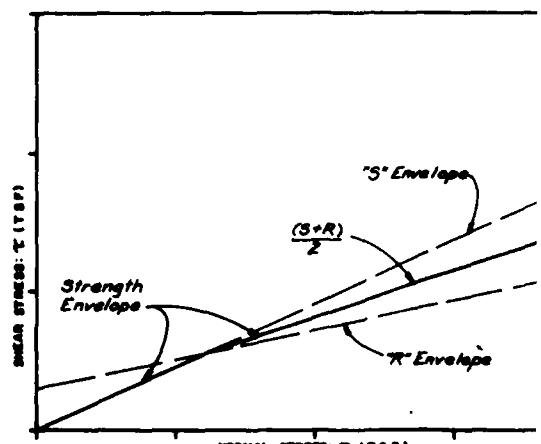
STAT…
ABUTMENT SECT…



STATION…
LEFT ABUTMENT SECTION…



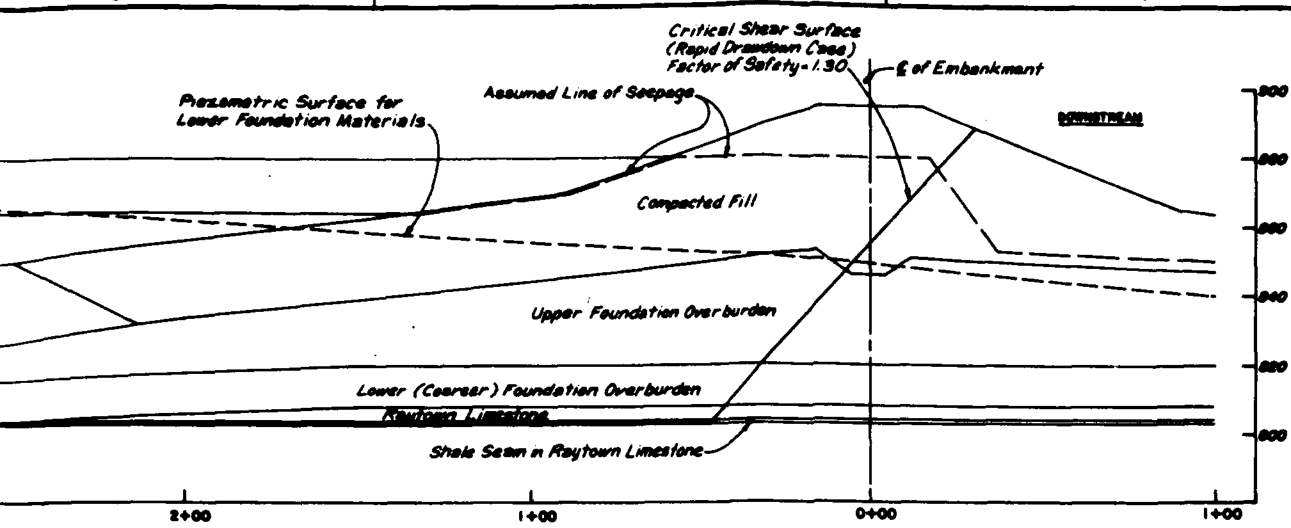
TYPICAL COMPOSITE ENVELOPE
(Used for rapid drawdown case)



TYPICAL INTERMEDIATE ENVELOPE
(Used for Partial Pool and Steady Seepage Ca…

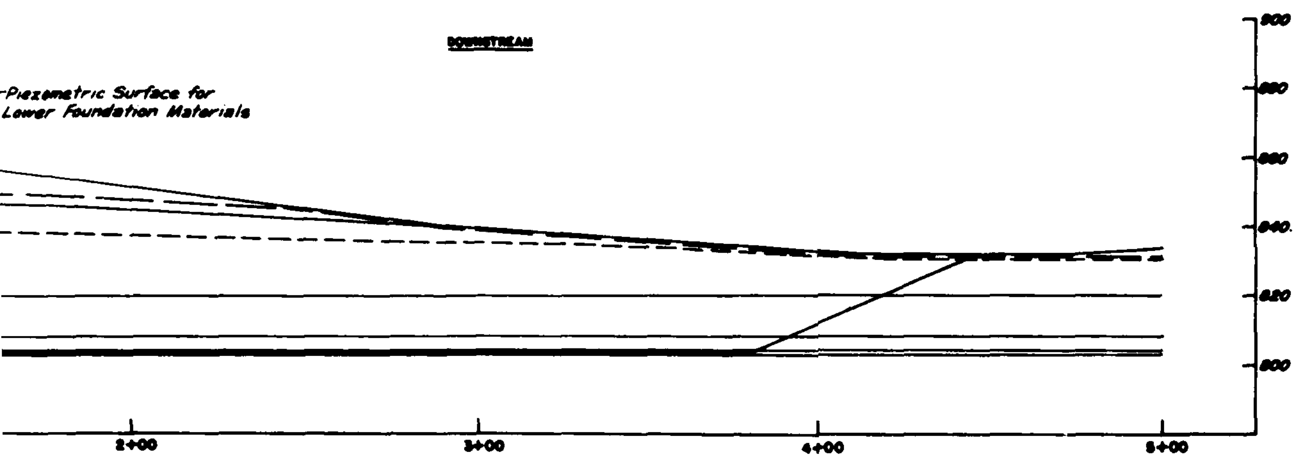
5 4 3

3 | 2 | 1



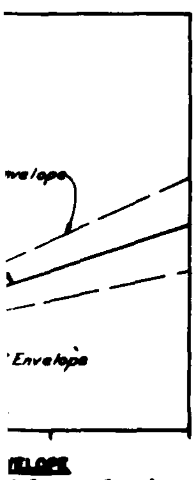
STATION 110+00

...UTMENT SECTION - RAPID DRAWDOWN CASE



STATION 110+00

...ENT SECTION - STEADY SEEPAGE CASE



...ELOPE

(...y Seepage Cases)

NOTES

1. SHEAR STRENGTH PARAMETERS FOR SOILS COMPRISING THE LEFT ABUTMENT ARE REFERENCED ON PLATE NO. 28.
2. SHEAR STRENGTHS FOR MOST OF THE MATERIALS WERE DERIVED FROM THE STRENGTH ENVELOPES SHOWN ON THIS PLATE. THE S STRENGTH WAS ASSIGNED TO THE LOWER (COARSER) FOUNDATION OVERBURDEN MATERIAL AND THE RESIDUAL STRENGTH WAS ASSIGNED TO THE SHALE SEAM IN THE RAYTOWN LIMESTONE.
3. THE ASSUMED LINE OF SEEPAGE IS USED TO DETERMINE WATER PRESSURES ACTING ON THE COMPACTED FILL AND BERM FILL MATERIALS.
4. THE PIEZOMETRIC SURFACE IS USED TO DETERMINE WATER PRESSURES ACTING ON LOWER (COARSER) FOUNDATION OVERBURDEN AND BEDROCK MATERIALS. THE PIEZOMETRIC SURFACE IS DERIVED FROM WATER LEVELS WHICH WERE READ IN A LINE OF PIEZOMETERS AT STATION 110+00 AFTER THE INSTALLATION OF REMEDIAL MEASURES AND PROJECTED TO THE REQUIRED POOL ELEVATIONS.
5. WATER PRESSURES FOR THE UPPER FOUNDATION OVERBURDEN MATERIAL ARE INTERPOLATED VALUES FROM THE ASSUMED LINE OF SEEPAGE AND PIEZOMETRIC SURFACE.
6. THE RAYTOWN LIMESTONE BEDROCK MEMBER IS SHOWN TO EXTEND 257 FEET UPSTREAM. THIS UPSTREAM LIMIT WAS DETERMINED FROM THE TOP OF BEDROCK CONTOURS AND SUBCROP MAP (SEE PLATE NO. A-9) AND FROM THE DRILL LOG DETAILING THE INSTALLATION OF PIEZOMETER P-110-2 (SEE PLATE NO. 12).
7. A COMPUTER PROGRAM ENTITLED UTEXAS2 WAS USED TO PERFORM THE STABILITY ANALYSES. OPTIONS WERE SELECTED WHICH ALLOWED THE PROGRAM TO SEARCH FOR THE SHEAR SURFACE CORRESPONDING TO A MINIMUM FACTOR OF SAFETY AND TO SOLVE FOR THE FACTOR OF SAFETY UTILIZING SPENCER'S PROCEDURE.

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: M.S.G.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT
Drawn by: V.A.S.	RAPID DRAWDOWN AND STEADY SEEPAGE STABILITY ANALYSES (LEFT ABUTMENT)
Checked by: J.B.H.	Scale: AS SHOWN
Submitted by: F.C.W.	MARCH 1987
	File No. RP-3-1780

SUPPLEMENT NO 1 PLATE NO. 30

3 | 2

A | B | C | D

APPENDIX A

OPERATION AND MAINTENANCE MANUAL

SMITHVILLE LAKE
LITTLE PLATTE RIVER, MISSOURI

APPENDIX V

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

SUPPLEMENT NO. 1

APPENDIX A

SMITHVILLE DAM
LEFT ABUTMENT STABILITY REPORT

July 1984

DEPARTMENT OF THE ARMY
Kansas City District, Corps of Engineers
Kansas City, Missouri

OPERATION AND MAINTENANCE MANUAL

SMITHVILLE LAKE
APPENDIX V
EMBANKMENT CRITERIA AND
PERFORMANCE REPORT

SUPPLEMENT NO. 1

LEFT ABUTMENT REMEDIAL MEASURES

APPENDIX A

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Drawings (cont.)

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1. Introduction.--Smithville Dam is located in northwest Missouri on the Little Platte River, one mile northeast of Smithville, Missouri. The project was authorized by the Flood Control Act of 1965 with construction starting in February 1974. Impoundment was begun in October 1979 but lake filling was delayed because of real estate acquisition problems. Multipurpose pool, elevation 864.2, was first reached in June 1982. The record high pool of elevation 869.4 was reached in April 1983 and again in May 1984. Flood control pool (full pool) is elevation 876.2. A spillway crest pool is elevation 880.2.

At the 2nd periodic inspection in the fall of 1982, which was the first inspection after the pool reached multipurpose level, high piezometric response was noted in several of the piezometers downstream of centerline. Comparisons between recorded levels and those anticipated during design were made. In general, it was noted that piezometric levels in the foundation under the downstream slope were somewhat above the horizontal pervious drain under multipurpose pool conditions. It was believed from reviewing the embankment design memorandum that piezometric levels for stability studies were assumed to be at the base of the pervious drain downstream of centerline. At that time it was believed the safety factor might be somewhat lower than the 1.6 shown in the embankment design memorandum for the steady seepage case. It was recommended that close monitoring of piezometric levels be continued, particularly at Station 110+00 where the level was about 3 feet above ground surface at the toe.

In April 1983, a site visit was made to inspect the dam with a record high pool at elevation 869.37. A seep area had developed at the toe at Station 110+00, however, seepage was not enough to observe flowing quantities. Shortly after this, work was begun on the embankment criteria report and review of stability analysis for the embankment DM was started. In August 1983 complaints from an adjacent landowner about a wet area 3,000 feet below the main dike were reported, and an investigation of seepage in the left abutment was initiated. As a result of this investigation, which included installation of additional piezometers through the downstream slope and a review of the original design stability analysis, it was found that:

a. The piezometric level assumed for design in the left abutment was actually some 20 feet below the base of the horizontal pervious blanket in this area.

b. Projected piezometric levels for a spillway crest pool based on recorded data to date were about 25 to 35 feet above what was assumed for design.

c. The higher than anticipated piezometric levels meant that available shear strength of the foundation shales became more critical.

d. The uppermost bedrock units in the left abutment have been exposed to erosional unloading and glacial loading. These factors, coupled with a nearly horizontal bedrock surface and essentially flat lying sedimentary rock units, suggested the possibility of the existence of a shear zone or zones near the bedrock surface.

e. Preliminary stability analysis conducted with the projected piezometric levels and the DM design strengths showed the safety factor well below the desired 1.6 of the D.M.

Accordingly, a more extensive stability investigation was initiated, and a revised plan of lake releases was quickly put into effect. The objective of the revised plan was to reduce the chances of subjecting the dam to high pool levels. The frequency of monitoring the dam was increased for all pool levels above multipurpose. Further, two inclinometer devices were installed in the most critical areas at Stations 108+00 and 110+00. The investigation included additional drilling and sampling, and installation of additional piezometers in the left abutment area. Since there were immediate concerns for the stability of the dam at high pool levels, an interim solution to improve stability was developed. It consisted of reducing piezometric levels at high pools by pumping test wells installed through the downstream slope. This action allowed time to complete the investigation and to develop a permanent solution or design measure which would assure the long term safety of the dam. Once the test wells were installed and operational, the project was returned to a normal operating plan. Following installation of the pumped test wells, four flowing test wells were installed at the toe of the embankment. The effect of these test wells were analyzed to determine if pressure relief wells at the toe would provide a satisfactory permanent solution.

2. Geology.--Smithville Lake is located near the southern limit of the Dissected Till Plains Section of the Central Lowlands Physiographic Province. Major topographic features are the maturely to submaturely developed valleys of Little Platte River, Crows Creek, and Camp Branch. Drainage patterns typical of northern Missouri are developed on thick glacial deposits resulting in gently rolling topography. Bedrock exposures are not common but can occasionally be found along the bases of valley walls of major streams. Maximum relief in the area is about 160 feet.

a. Glacial history.--Pleistocene glaciers extended into this region of Missouri approximately 750,000 years ago during the Kansan glacial episode and persisted for approximately 100,000 years. Glaciers may have also advanced into the area during the earlier Nebraskan episode. Both the Nebraskan and Kansan advances were from the north-northwest and are attributed to the Iowa ice lobe from the Keevatin ice center in Canada. Since the same general regions were traversed during both episodes, the content of resultant drift materials is similar and difficult to distinguish. The southern limit of glaciation is generally recognized as being slightly south of and approximately parallel to the present course of the Missouri River.

Pleistocene ice sheets have been compared in size and extent to those of the Antarctic which have an average central thickness of about 6,500 feet. Estimated thicknesses of marginal masses are of the order of 1,600 feet. Glacial erosion was primarily by abrasion and quarrying whereby slabs of frozen ground were sheared from and dragged forward over nonfrozen ground. Magnitudes of erosion were dependent upon the thickness and velocity of the ice mass, the nature of materials incorporated into the basal ice, and the character of surfaces overridden. Glacial sediments include nonstratified till and, less frequently, fluvio-glacial deposits of stratified silts, sands, and gravels. Drift of variable thickness has been deposited upon essentially

flat lying Pennsylvanian bedrock and is the thickest in pre-Pleistocene topographic lows.

b. Overburden.--Overburden in the vicinity of the dam is of two principal types; alluvium and glacial drift. Alluvium occupies the valleys of the Little Platte River and its tributaries and generally consists of lean and fat clays overlying clayey sands and sandy clays with minor amounts of basil gravel. Thicknesses range from 25 to 50 feet. Upland areas are mantled with deposits of glacial drift. In the left abutment area, the drift ranges in thickness up to 85 feet and generally consists of 20 to 60 feet of till overlying 5 to 25 feet of coarser outwash sediments. Till, in general, is composed of unsorted, unconsolidated, nonstratified sediments deposited directly by and underneath glacial ice masses and consists of heterogeneous, random mixtures of clay, silt, sand, gravel, cobbles, and boulders. Till in the left abutment is predominantly fine-grained, relatively impervious silty clay and clayey silt with scattered gravel and cobbles and isolated sand lenses. Coarser materials underlying the till are meltwater sediments deposited beyond advancing or retreating ice sheets which form a permeable zone of saturated dirty sand, gravel, and cobbles. This zone is exposed near the base of the valley walls in the reservoir area just upstream of the dam and therefore, is subjected to reservoir hydrostatic pressures. The overlying less permeable fine-grained silty clays and clayey silts create a confined flow system. Seepage can potentially occur where the piezometric surface in the confined system is above the ground surface. Plate A-3 shows where these conditions exist.

c. Bedrock.--Near surface bedrock strata are of the Pennsylvanian System, Lansing and Kansas City Groups and consist of alternating beds of shale and limestone. The uppermost units within the area of this investigation are, in descending order, Raytown Limestone, Muncie Creek Shale, Paola Limestone, Chanute Shale, and Cement City Limestone. A geologic column for the left abutment is shown on plate A-3A. The top of bedrock surface in the left abutment landward of approximately dam Station 107+50 is formed by the Raytown Limestone. Top of bedrock contours and subcrop map are shown on plate A-5.

The configuration of the left abutment bedrock surface is the result of a pre-Pleistocene stream channel trending generally east-west through the abutment. It is one of two major channels mapped in the reservoir area which are associated with the ancestral Missouri River drainage system prior to the advance of Pleistocene glaciers. The other is located several miles upstream of the dam in the reservoir area. As ice masses traversed the area, existing sediments were scoured away and near-surface bedrock strata subjected to shear forces induced by ice thrusts.

3. Design criteria.

a. Design measures, left abutment.--In Design Memorandum No. 10, Smithville, "Soil Data and Embankment Design," high piezometric levels resulting from underseepage were not expected to be a problem "due to the thickness of the impervious foundation materials, the large amount of fines in the sands and gravels and the scarcity of continuous pervious layers beneath the embankment." It was thought the horizontal pervious drain would "intercept groundwater seepage from the foundation." However, precautionary

measures were taken. Random fill was placed upstream of the impervious core from Station 110+00 to the end of the dam and covered an existing draw on the upstream slope. This draw was believed to possibly intercept zones of sand and gravel which might be continuous through the abutment. A 5-foot thick clay blanket was placed on the left abutment starting at Station 117+00 and extending to elevation 880 (see plate A-1). It completed cover over the draw on the upstream slope. A centerline cutoff trench to rock was not excavated because of relatively thick overburden. However, curtain grouting was conducted through the overburden from about Station 105+00 to 109+00. Grouting extended through the Raytown, Muncie Creek, and Paola units into the Chanute Shale (see plate A-6). As part of the embankment construction, a similar draw downstream of the embankment starting high up on the abutment and trending back towards centerline was backfilled with random material.

b. DM stability.--Water pressures during the design analysis were computed based on the assumption of hydrostatic pressure below the saturation line. For the left abutment section, the saturation line under the downstream slope was in the foundation at elevation 825.

For the original embankment design, a minimum safety factor of 1.6 was sought when both the partial pool and steady seepage cases used a peak strength approach and when the shale-overburden contact formed part of the failure surface. When a residual shale shear strength approach was used, a safety factor in excess of 1.0 was sought. Bedrock was assumed to be shale in all cases of the DM stability analysis. The higher strengths for overburden or limestone, where present, were not used since "(1) there may be remnants of weathered shale above the limestone; and (2) it would require thin, partially weathered, jointed limestone to carry very large forces."

The stability analyses were conducted using a computer program entitled "Stability Analysis-Wedge Method" (File No. 41-R3-C102), commonly referred to as KC SLOPE. KC SLOPE is capable of searching for a minimum safety factor for a wedge-shaped failure surface. The program accounts for water forces assuming hydrostatic pressure beneath the assigned saturation line. Analyses were conducted in accordance with EM 1110-2-1902 (1968 draft) with the slope of $E_A = .08$, and $Ep = 0.0$. Slope geometry is shown on plate A-26. DM design strengths and soil properties are shown on plate A-27.

Results of the steady seepage case with the failure surface occurring at the top of bedrock are as follows. A safety factor of 1.64 was obtained using peak design strengths for the left abutment section (approx sta 110+00). When residual design shale shear strengths were used, the safety factor dropped to 1.08.

4. Left abutment embankment-stability analysis.--Reanalysis of the steady seepage case of embankment stability was conducted for the left abutment section as a result of actual piezometric levels higher than those assumed during design. In general, these piezometric conditions are characterized by 1) above ground uplift levels in the foundation at the downstream toe near the left abutment; 2) unusually high centerline piezometric levels; and 3) piezometric levels in the foundation that respond from 40 to 60 percent to pool changes with little time lag (see plates A-9 to A-14).

a. Method of stability analysis.--The computer program used in the DM analysis, KC SLOPE, was not written to account for uplift forces in the foundation. For this reason, all subsequent analyses were conducted with the hand wedge method in accordance with EM 1110-2-1902 (April 1970) with the slope of $E_A = E_P = 0.0$ (since ab > 7/8H) and with a computer program, SLOPE8R, developed at the University of California at Berkeley. This computer program was used because it can analyze a non-circular failure surface and boundary water pressures can be assigned for the slide surface. SLOPE8R uses Spencer's procedure to calculate the safety factor for specified non-circular slip surfaces. It is a special solution of the Morgenstern and Price method in which all the interslice side forces are assumed to have the same inclination. The program satisfies both force and moment equilibrium conditions for each slice. The two unknown parameters, F (the safety factor) and theta, θ, (the side force inclination) are varied simultaneously by iteration until a convergent solution is found with the net force and moment imbalance less than specified values. The method will not compute the same safety factor nor locate the same minimum failure surface as the hand wedge method. In satisfying moment equilibrium it usually results in solving for a different side force inclination, θ, than is assumed in the hand wedge method. In all cases, the safety factor obtained through hand wedge studies were lower than those obtained by SLOPE8R, because side force inclination was greater than that assumed in the hand wedge analysis. When the same side force is used in the hand wedge analysis, computed safety factors agree quite closely. This should be expected because both methods would then be solving for F by force equilibrium equations.

b. Preliminary analysis.--Preliminary analysis of the steady seepage case of left abutment embankment stability was conducted at Station 110+00, because recorded piezometric levels in the foundation were some 25 feet higher than were assumed in the design analysis. Stability studies used full pool phreatic conditions in the embankment portion of the slide surface and piezometric uplift levels projected for full pool conditions in the foundation portion of the slide. Projected levels were based on the recorded responses of P-110-3 and P-110-4 to changes in pool elevation. Projected uplift levels were elevation 859 at 20 feet upstream of centerline, with a straight line gradient to elevation 849.5 at 320 feet downstream of centerline. The foundation was assumed to be saturated to the base of the horizontal pervious drain.

Safety factors calculated using the projected uplift pressures on the slide surface and peak DM design strengths ($c = 0.0$, $\tan \phi = .30$ for shale) were 1.42 using SLOPE8R with = 8.0 degrees and 1.23 using the hand wedge method. When the shale shear strength was reduced to residual DM design condition ($c = 0.0$, $\tan \phi = .16$), the safety factors decreased to 1.0 using SLOPE8R with $\theta = 6.9$ degrees and 0.92 and by the hand wedge method. DM design strengths are shown on plate A-27. A summary of stability analyses is shown on plate A-28.

5. Field investigations.

a. Additional sampling rationale.--Shale design strengths in the DM, both peak and residual, were based on laboratory tests from samples obtained from the shale units in the right abutment, outlet works area, and valley. Difficulty was encountered obtaining shale samples from the left abutment because of the thick overburden, the presence of gravels and cobbles above rock, the thinness of the shale seams, and the weathered, broken nature of the limestones overlying the shales. However, the strength of these same shale units in the outlet works did not control the selection of the design strength envelopes. Since the safety factor of the stability analysis for the left abutment was adequate, additional sampling and testing for the left abutment was not warranted for the original design.

The design shale shear strengths became more critical when higher than anticipated piezometric levels were recorded in the left abutment. The two abutments have different geologic histories. The upper units in the rock foundation of the left abutment have been subjected to both erosional unloading and glaciation, whereas the same units in the outlet works area and right abutment were protected by overlying bedrock units. It was believed either weathering and movement of the ice mass over the left abutment or valley stress relief could have caused one or more shear zones near the bedrock surface. The strength of a shear zone could be less than the design strength.

Thus, an investigation was initiated to determine if a weak zone or zones did exist. Sampling efforts were directed towards obtaining 6-inch core samples of soft shale seams in the Raytown Limestone, the contact of the Raytown and underlying Muncie Creek shale, and suspected soft seams in the upper Muncie Creek where persistent core losses had occurred in previous borings. It was also desired to determine if soft zones, shear planes or slickensides were present in the remainder of the Muncie Creek Shale or in the underlying Chanute Shale. Drilling and sampling was also conducted near Station 105+50 where it was suspected soft weathered Muncie Creek might be present because of the absence of the overlying Raytown Limestone in this area. Soft, highly weathered Muncie Creek had been excavated in the right abutment cutoff trench during construction.

b. Exploratory drilling and instrument installation.--The exploration program included a series of core borings to obtain shale samples for shear testing and installation of inclinometers to monitor embankment movements and piezometers to define pressure gradients and monitor piezometric responses to pool fluctuations. Subsurface information was obtained from explorations for this stability study and previous seepage investigation, and from preconstruction borings. A plan of explorations for the left abutment is shown on plate A-3. The plan shows instrumentation currently being monitored in the area of concern as well as locations of core borings done for this study.

Sixteen exploratory borings were completed as part of this study. Nine were 6-inch diameter core borings into bedrock which were completed as either piezometers in the overburden or as pore pressure devices in the foundation shales. Six borings were drilled just through the overburden and completed as piezometers. Overburden piezometers were constructed with the

screened interval set in the more permeable basal dirty sand, gravel and cobbles. Inclinometer casing was installed in two borings. Seven of the 6-inch core borings were advanced through the Chanute Shale and terminated in the Cement City Limestone, one was advanced through the Paola Limestone and terminated at the top of the Chanute Shale, and one was advanced 5 feet into and terminated in the Raytown Limestone. Selected samples of the shale seam in the Raytown Limestone, the Muncie Creek Shale, and the Chanute Shale were preserved for shear testing. The first three core borings completed contained several thin zones of core loss and spins in the Muncie Creek and Chanute Shales. In order to determine if these resulted from the drilling process or represented other weak zones, a method was developed whereby an NX size core hole was drilled through the Chanute Shale, filled with cement grout, and the grout allowed to cure for approximately 60 hours. This hole was then overcored with a 6-inch core barrel. If thin weak zones had been present, the rigid column of grout would have provided sufficient torsional shear resistance to prevent spins and core losses. This procedure was used in C-528 which is located only 10 feet south of previously drilled C-527, in order to be able to recover and inspect a complete section of core. Core from the NX hole, from the 6-inch overcore and from C-527 were physically compared to determine if spins and core losses represented shear zones. In fact, unsheared shale was recovered in the 6-inch overcore of C-528 at the horizons where spins and core losses occurred in C-527 and NX portion of G-528.

Overburden drilling was accomplished with 9-7/8-inch rockbit or 9-1/2-inch fishtail bit except in UC-530 and UC-531 which were drilled in an area suspected to be underlain by weathered Muncie Creek Shale. In these two holes, the lowermost 12 feet of overburden was sampled with a 5-inch, fixed piston Shelby tube sampler in order to accurately locate the top of bedrock and to insure a quality sample of the contact.

Inclinometer borings were drilled 5 feet into the Cement City Limestone with a 5-7/8-inch rock bit and completed with 3-inch grooved aluminum inclinometer casing grouted the full depth. The lowermost 5 feet of each hole was grouted with 1:1 water:cement grout to anchor the casing firmly into the Cement City Limestone. The remainder of the annulus between the casing and drill hole was filled with grout consisting of 50 percent bentonite by weight of Portland Cement. A basic mixture consisted of a 94 lb. sack of Portland cement, 47 lb. of powdered bentonite and 38 gallons of water. This mixture forms a semi-rigid material capable of proportionally transmitting any deformation to the casing should movement occur.

Piezometers and pore pressure devices are the open tube type and are constructed of 3/4-inch PVC pipe with 1-1/2-inch diameter, 1-1/2 ft. long 0.020-inch slotted PVC well screen tips. Overburden piezometer tips are isolated and sealed in the lower, more permeable materials. Five open tube pore pressure devices were installed in the foundation shales; two in the thick shale seam in the Raytown Limestone, two in the Chanute Shale and one in the Muncie Creek Shale.

c. Results of exploratory drilling program.--At the start of the investigation the purpose of the drilling program was primarily to sample and preserve for testing, the Raytown-Muncie Creek contact. It seemed then that the most likely plane to have been disturbed by ice thrust deformation would

have been the Raytown-Muncie Creek contact since the Raytown forms the top of bedrock and consists of moderately hard limestone directly overlying a relatively soft shale. The first core hole, however, revealed slickensided planes in a very soft shale seam in the lower part of the Raytown Limestone. The seam is about 0.4 to 0.5 feet thick, approximately 1-1/2 feet above the Muncie Creek and is persistent throughout the abutment. Apparent shear planes were observed in every sample of the shale seam recovered. Several thin shale partings are present above this seam but they are not continuous. The contact of the Raytown with the Muncie Creek as well as the remainder of the Muncie Creek and the Chanute were determined to be inconsequential with respect to embankment stability.

d. Instrumentation monitoring.--Prior to the initiation of the seepage investigation, the basal layer the left abutment overburden was monitored by piezometers located upstream of centerline and at the toe at Station 110+00, and at the toe at Station 114+00 and Station 119+00. Monitoring of these piezometers was done on a quarterly basis. During the seepage investigation, additional devices were installed at the top of the dam at Station 106+00, downstream mid-slope at Station 110+00, downstream of toe at Station 110+00 and at the toe at Station 118+00. The devices confirmed the high pressures measured by the existing devices and enabled better definition of the piezometric surface. Monitoring was increased to monthly.

Recorded data showed piezometers in the left abutment foundation overburden respond rapidly to changes in pool level. Responses ranged from 40-60 percent with maximum time lag of a few days. All new installations responded similarly to existing devices (see plates A-9 - A-11A).

Beginning in April 1984 and as a result of the revised operation plan for the project, instrumentation was monitored according to the following schedule:

Pool Elevation	Frequency
864-868	weekly
868-869	biweekly
869 and above	daily

Monitoring included all piezometers, inclinometers, and alignment monuments in the left abutment area. This increased monitoring also resulted in better definition of projections of piezometric levels for a spillway crest pool condition (see plates A-12 - A-14).

Two inclinometers installed in April 1984 at Station 108+00 and Station 110+00 have shown no movement. They have been read at least weekly since installation with biweekly and daily readings at higher pool levels. Typical movement versus depth and movement along the Raytown shale seam versus time plots are shown on plates A-15 - A-16. Alignment monuments similarly have shown no indications of movement. Typical plots are shown on plates A-17 - A-18.

6. Test wells.--Seven test wells were installed in the left abutment area and tested to determine their effect of lowering the piezometric surface. Locations are shown on plate A-3. Three of the wells; W-1, W-3, and W-4; are

located in areas within which the piezometric surface is below existing ground surface and therefore, must be pumped. The other four; W-2, W-5, W-6, and W-7; are in the area within which the piezometric surface is above the ground surface and consequently, are free flowing.

a. Pumped test wells.

(1) Installation and development.--Pumped test wells were drilled with nominal 13-inch diameter rockbits and self-destroying organic polymer drilling fluid through the Raytown Limestone. Installation details are shown on plate A-19. Screen is 6-inch diameter type 304 stainless steel, continuous slot design. Casing is 6-inch diameter schedule 80 PVC pipe. Screen slot opening is 0.060-inch. Gravel pack gradation is as follows:

Sieve Size	% Retained
No. 4	12
No. 8	66
No. 16	90
No. 30	96
No. 50	99
No. 100	99.7
No. 200	100

Well development consisted of dispersing clays with a polyphosphate solution, surging with surge plunger, and high velocity water jetting through the screened section.

(2) Pump testing.--W-1, W-3 and W-4 were pump tested individually as they were completed. In addition, a long term pump test was done simultaneously pumping W-1, W-3, and W-4. W-1 was pump tested individually at 10 GPM while monitoring P-110-3, P-110-8, P-110-4, P-110-6, P-110-7, P-106-4, P-118-1 and P-119-1. No measurable responses were recorded in P-106-4, P-118-1 or P-119-1. Computed values for transmissivity (T) and storativity (S) from pumping W-1 are as follows:

P-110-3
T = 1000 GPD/ft.
$S = 1.4 \times 10^{-4}$

P-110-4
T = 1300 GPD/ft.
$S = 0.9 \times 10^{-5}$

P-110-8
T = 300 GPD/ft.
$S = 0.8 \times 10^{-4}$

P-110-6
T = 1800 GPD/ft.
$S = 0.8 \times 10^{-4}$

W-3 was pump tested at 25 GPM while monitoring P-114-1, P-113-1, P-112-1, P-110-3, P-110-8, P-110-4, P-110-6, P-110-7, P-118-1 and P-119-1. Measurable responses were recorded only in P-114-1 and P-113-1. P-112-1

was not functioning during the test. Computed valves for T and S from pumping W-3 are as follows:

P-113-1
T = 3200 GPD/ft.
S = 0.5 X 10^{-2}

P-114-1
T = 4300 GPD/ft.
S = 0.5 X 10^{-2}

A pump test was started in W-4 at 10 GPM while monitoring P-110-9, P-109-1, P-110-3, P-108-2 and W-1. However, after 2 hours pumping, drawdown in the well was near the top of the screen and no responses were observed in any of the piezometers. The test was terminated and the well was redeveloped. The well was not pump tested individually again.

The rather wide range of T & S values obtained from the pump tests and the somewhat unpredictable effects of pumping each of the three wells indicate significant lateral variation in permeability of and/or thickness of the basal pervious material. The transmissivity of the pervious appears to be greatest upstation of approximately Station 112+00 and is quite small downstation of approximately Station 108+00. The higher transmissivity in the vicinity of W-3 may be caused by either a thickening of the pervious zone or the presence of higher permeability material or a combination of both. The lower values near W-4 appear to be caused by the presence of less permeable material in this area. As shown on plate A-20, the greatest drawdown over the largest area results from pumping W-1.

On 31 May 1984, a long term pump test was started with all 3 wells pumping simultaneously. W-1 was operated at an average discharge of 7.5 GPM, W-3 at an average discharge of 17 GPM and W-4 at an average discharge of 5 GPM. Again, drawdown in W-4 was rapid with no corresponding response in nearby piezometers. The pump was shutoff, removed and the well again redeveloped. No improvement in well performance was realized. The test was run for 27 hours. All piezometers installed in the basal pervious overburden were monitored during the test. Distance-drawdown plots were constructed for W-1 and W-3 from pump test data at 24 hours and these contoured as cones of heads. Drawdown contours are shown on plate A-20. These were subtracted from contours of the piezometric surface projected to the spillway crest pool for the interim analysis and the resulting contours of the piezometric surface with pumping are shown on plate A-21. These piezometric projections were based on early data and were subsequently revised based upon additional data, spring 1984.

b. <u>Test relief wells</u>.

(1) Installation and development.--Flowing test wells were drilled with nominal 10-inch diameter rockbits. Because piezometer levels were above ground, weighted bentonite slurry drilling fluids were used. Installation details are shown on plate A-19. Screen is 4-inch diameter type 304 stainless steel continuous slot design. Casing is 4-inch schedule 80 PVC pipe. Screen slot opening is 0.060-inch except W-5 in which slot opening is 0.030-inch. Gravel pack gradation is the same as for the pumped wells. Development was done by jetting a polyphosphate solution through the screened section and by surging with a surge plunger.

(2) Well effectiveness.--Installation of 4 flowing test wells, W-2, W-5, W-6 and W-7 was completed on 9 June 84. After all the test wells were completed and developed, they were all plugged, the piezometric surface allowed to stabilize and then the wells unplugged. Piezometers were monitored daily for the first full week after unplugging the test wells and weekly thereafter. The pool was at elevation 868.4 during this time. Total drawdown in each piezometer in response to the flowing wells was plotted, contours drawn, and these values subtracted from refined piezometric contours projected to a pool at spillway crest for the final analysis. These are shown on plate A-22.

7. Shear strength testing.--To provide design strength parameters for the interim and final analyses, consolidated-drained direct shear "S" tests, residual shear tests, and triaxial compression consolidated-undrained (with pore pressure measurement) "R" tests were conducted.

a. Test procedures.

(1) Direct shear and residual shear.--The direct shear and residual shear tests were conducted with a 3-inch square shear box in which a usually 0.5-inch thick sample is made to shear horizontally. For intact shale specimens the peak strength is obtained within .3 inches of horizontal displacement. The residual shear condition is attained by repeatedly reversing the direction of shear on the induced shear plane. Intact, precut and remolded specimens were tested. Specimens were consolidated in most cases to 6.0 tons per square foot (TSF) prior to shearing. Deformation rates were on the order of 0.2 to 0.4 inches/day. Soft shale specimens were trimmed using a band saw; a diamond rock saw was used when harder rock formed part of the specimen.

(2) Triaxial compression consolidated-undrained (with pore pressure measurement) R tests.--The R tests were conducted to determine total and effective stress strength parameters on slickensided surfaces in the soft Raytown shale seam. The slickenside was oriented at 55 degrees to 60 degrees from horizontal to assure failures developed on the slick. Specimen size was 1.4-inch diameter by 3-inch long so that 2 specimens could be trimmed from a single 6-inch core containing the near horizontal slick. Specimens were trimmed using a band saw and a supporting jig to prevent disturbance or shearing on the slick. All specimens were back-pressure saturated to obtain 100 percent saturation (Skempton's B parameter greater than .95 was required) prior to consolidation and shear. The shearing rate was selected to allow pore pressure measurements and was based on t_{50} from the consolidation data.

b. Initial tests.--A 4-inch core sample (hole I-108-1) recovered from the Raytown Limestone contained soft shale seam about 1-1/2 foot above the Muncie Creek contact. The seam had a continuous near horizontal slickensided surface but the sample was too small to test intact. There was, however, sufficient material to run a remolded specimen. The specimen was consolidated to 12 TSF, rebounded to 6 TSF and precut prior to shearing. The specimen developed a residual shear strength of tan ϕ_r = 0.147. At first this was believed to represent a lower bound residual strength. However, subsequently it was discovered that the shear surface contained "gritty" particles and the test results were considered unrepresentative.

The first 6-inch core hole (C-525) recovered the same thick shale seam in the Raytown, but as in the earlier exploratory drilling efforts, core loss and spins occurred in the upper Muncie Creek. The slick was present in the thick Raytown seam (sample 1) and direct shear and residual shear tests were conducted. A tan ϕ = .416 and tan ϕ_r = .196 were obtained. The Raytown/Muncie Creek contact as well as other soft seams in the Raytown and a low angle fracture in the Chanute shale were tested. These results are presented on plates A-23 - A-24. Based on these encouraging results, it was felt the suspected soft seams in the Muncie Creek which had not been successfully recovered and possibly other areas of core loss would probably dictate the design shear strength. Sampling efforts were adjusted accordingly. However, it was considered desirable to obtain at least one other sample of the Raytown slick to better define the effective stress envelope, especially any cohesion intercept, and to develop a total stress strength envelope in case this was needed for the stability analysis.

c. Definitive testing.--An R test was conducted on the Raytown slick (C-527). Although problems were encountered with the test, an interim design strength of c' = 160 pounds per square foot (psf), tan ϕ' = .265 was selected for the Raytown seam see plate A-27. Since this was considerably lower than the previous direct shear result from C-525, another direct shear test was run C-528. It did not confirm the earlier direct shear test results. This time a tan ϕ= .289 and tan ϕ_r = .129 was obtained. The inconsistency is best explained by the concave shape of the slick surface in the earlier test as opposed to a near planar surface in the latter. Because of the wide difference in results, it was considered prudent to run an additional R test and residual shear tests to increase the confidence level in the strength results. In the meantime, by using an overcoring technique, it was determined that suspected soft seams in the upper Muncie Creek were not present. Thus the design strength would be dictated by the slickensided surface which had been encountered in all the samples recovered from the Raytown shale seam.

Residual strengths obtained on precut specimens from the Raytown seam were nearly identical to the result from the intact specimen on the planar slick, tan ϕ_r = .13. Of some interest is the fact that the peak strengths on the precut surfaces were also quite similar to the peak strengths on the planar intact specimen. The second R test result from C-532, was c' = 500 psf, tan ϕ' .31. A final direct shear test was run at a consolidation pressure of 4 TSF. From these additional tests, the final design strength was adjusted to c' = 250 psf, tan ϕ' = .268 (see plate A-27). Thus the selected peak design strength was not significantly lower than the original design peak envelope.

8. Stability analysis criteria.

a. Shear strength considerations.--The DM and preliminary stability analyses were conducted using two different strength approaches: 1) peak design strength along the entire failure slide surface and 2) peak design strength along the failure surface in the active and passive wedges and residual shale shear strength in the central block portion of the slide. These strength approaches were discussed with MRD and OCE personnel at a site inspection in late April. It was agreed that use of the residual strength in a stability analysis may be overly conservative. However, use of peak strengths in the analysis was probably unconservative, because of strain

incompatibility between a shear zone in the shale, the foundation overburden and the embankment. With small strains in the shale, the peak strength could be developed before peak strengths are attained in the embankment and overburden.

Accordingly, a third approach was considered for the interim and final analyses which would allow use of peak strength in the active wedge portion of the failure surface and in the shale seam, but not in the passive wedge portion since relatively large displacements would be required to develop full passive resistance. It was considered reasonable to use strengths in the passive wedge (foundation overburden) which correspond to 0.5 percent strain development. Strains somewhat larger than this were required to develop the peak strength in tests on the slicksided shale surface. Test results on overburden clay samples from the left abutment and for the embankment were available from the earlier design investigation. Although record control tests from the embankment indicate higher strengths than those used for the design analysis, design strengths were not changed. (The DM strengths were determined for minimum required placement conditions, 95 percent maximum density and +3 percent optimum water content.) As a result of the extensive exploratory boring through the foundation overburden, it was concluded that a coarse-grained pervious layer was consistently present immediately above rock. Since DM tests on overburden samples were on the weaker lean and fat clays it was believed reasonable to assign a strength to the pervious layer of $c=0$, $\phi=30$ degrees in the active wedge portion of the failure surface. In the final analysis, the Raytown Limestone above the shale seam in the active wedge was assumed to have a vertical joint and was given no shear strength. See plate A-27 for a summary of the design strengths used for the various analyses.

b. Required safety factor.--In many cases during a DM stability analysis, comparatively little is known about specific foundation conditions at a given location. The design safety factor of 1.5 required by EM 1110-2-1902 for the steady seepage case is in part, intended to provide an added degree of safety, in case possible locally weak areas do exist. Since considerable exploratory work, sampling, and testing was conducted for this investigation, a safety factor of 1.3 for the left abutment embankment is appropriate for the detailed stability analyses conducted.

The minimum safety factor of 1.3 must be obtained using the more conservative hand wedge method (with horizontal E_A and E_P forces) instead of SLOPE8R. The hand wedge method has been used extensively by the District in past stability analyses and thus is an integral part of our slope design experience. SLOPE8R was used to expedite the analyses and to find the most critical combination of slope and piezometric levels. The results were checked by the hand wedge method.

9. Left abutment embankment stability analysis.--The re-analysis of the left abutment section for embankment stability was conducted in three phases. A preliminary stability analysis used DM strengths and projected piezometric uplift levels based on recorded data. The results of the preliminary analysis led to concerns for the stability of the dam at high pool levels. Subsequent analyses included an interim analysis for an interim solution and final analysis for a permanent solution.

a. Interim analysis.--An early objective of the investigation was to determine whether an interim solution could be effected which would not require the construction of an expensive stability berm. Further, emergency construction of a berm would have in all likelihood been rockfill and made additional and badly needed exploratory work very difficult, if not impossible.

The interim solution which was discussed with MRD in June 1984, consisted of pumped test wells installed through the downstream slope. The interim analysis was conducted at Station 110+00 with pool levels at multipurpose pool and at spillway crest pool. The third strength approach was used for the analysis, using peak strengths in the active wedge and central block and strength corresponding to 0.5 percent strain in the passive wedge. The strengths used are shown on plate A-27.

Piezometric levels used corresponded to recorded values for multipurpose pool and projected values for a spillway crest pool condition. Drawdown obtained from the pumped test well data at multipurpose pool was subtracted from the projected piezometric levels at spillway crest. The piezometric contours used in the interim analysis are shown on plate A-21. Results of these studies are summarized below and on plate A-28.

	Safety Factor		
	Hand Wedge	SLOPE8R	
Multipurpose Pool, El. 864.2	1.40	1.66	(θ=8.5°)
Spillway Crest Pool, El. 880.2	1.20	1.41	(θ=8.0°)
Spillway Crest Pool with pumped test wells	1.32	1.55	(θ=8.9°)

From the plot of safety factor versus pool level it was determined that pumping would begin whenever a pool of elevation 872 or higher is forecast to assure a safety factor of greater than 1.3 (see plate A-29).

While this interim solution provides an adequate safety factor, it is not desirable as a permanent solution since it requires a specific human response for the life of the project whenever high pools are experienced. There are concerns that at some time in the future, when current personnel are no longer around, that the necessity of maintaining and of pumping the test wells each time the pool approaches elevation 872 may be forgotten. The solution is also dependent on complete mechanical and electrical reliability of pumps and generators; any significant downtime during periods of high pool would be potentially dangerous.

b. Final analysis.--The final stability analysis consisted of locating the most critical slope in the abutment area for the steady seepage case at spillway crest pool, both with and without pressure relief wells at the toe. Piezometric levels were projected for this pool condition, using the most recent recorded piezometric levels. This analysis of the more recent piezometric data indicated some reduction in anticipated pressure levels was justified. Drawdown data obtained from the installation of 4 test relief wells near Station 110+00 was used to evaluate the effect on stability. The drawdown obtained with the pool at elevation 868 was applied directly to refined projected spillway crest piezometric levels (see plate A-22).

With the completion of laboratory shear strength testing, the design shear strength for the shale foundation was further refined as discussed previously. This final design shear strength was used with the same strength approach used in the interim analysis, that is, peak strengths in the active wedge and central block, strength at .5 percent strain in passive wedge and a required safety factor of 1.3 (see plate A-27). Results at Station 110+00 are as follows:

	Safety Factor	
	Hand Wedge	SLOPE8R
Spillway Crest Pool, without relief wells	1.25	1.47 (θ=8.7°)
Spillway Crest Pool, with relief wells	1.30	1.53 (θ=8.7°)

The safety factors above are conservative in that piezometric levels assumed for the "with wells" condition are conservative. The effect of the wells at a spillway crest pool should be greater than with the pool at elevation 868.

Further the safety factor will be increased by lowering the outfall of the wells with the installation of a buried collector pipe at a depth of 3-4 ft. and installing more pressure relief wells at the toe and through the embankment with the "outfall" in the pervious drain. (See recommendations.)

10. Conclusions.--The following conclusions are made.

a. The embankment on the left abutment as constructed has an inadequate safety factor with respect to slope stability for a spillway crest pool, steady seepage case, because of higher than anticipated piezometric levels in the foundation.

b. An interim solution of pumped test wells through the downstream slope will provide an adequate safety factor until a permanent solution can be implemented. The pumping will commence whenever the pool is predicted to rise to elevation 872 or higher.

c. Construction of a stability berm is not warranted. In addition to the cost ($500,000 ±) and environmental impacts, it cannot be implemented as quickly or as efficiently as pressure relief wells. Installation of pressure relief wells at the toe area will provide an adequate safety factor for spillway crest pool, steady seepage conditions.

11. Recommendations.--The following recommendations are made.

a. Pumped test wells should continue to be implemented as the interim solution until the permanent solution is in place.

b. The following schedule for monitoring of the existing instrumentation in the left abutment area is recommended.

Pool Elevation	Frequency
865.5 and below	monthly
865.4 to 869	weekly
869 - 872	biweekly
872 and above	daily

c. Additional pressure relief wells should be installed in the toe area as shown on the plate A-30. The well outfalls should be connected to a buried collector pipe. Additional wells should be installed through the downstream slope with a riser section containing a well screen at the pervious horizontal sand blanket to provide pressure relief. Estimated cost for the above remedial measures are as follows:

12 additional pressure relief wells @$6,000 ea	$72,000
Buried collector system	8,000
	$80,000

d. It is also recommended that the wells be installed with the District's own hired labor drill crews for the following reasons:

(1) With the existing piezometric surfaces well above the ground surface in the area where the wells for a final solution would be installed, the risk of an inexperienced and/or inept contract drill crew losing a hole or otherwise compromising the integrity of the dam is too great.

(2) The Kansas City District drill crews have successfully and efficiently installed four test wells in this area without any compromise of the safety of the dam.

(3) Due to the heterogeneity of the subsurface conditions in the left abutment area, some adjustments in well location, screen locations, screen size, etc., may be required based on the conditions encountered. District drill crews can more readily adapt to such required changes.

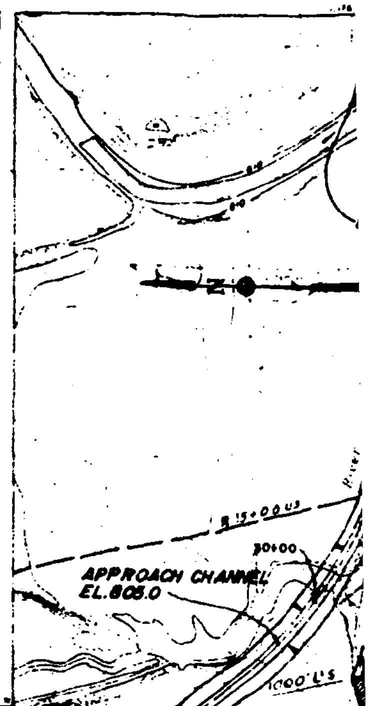
(4) If funds were made available for implementation of the proposed permanent well solution, District drill crews could have all of the work completed by time it would be put out for bids under normal procurement procedures.

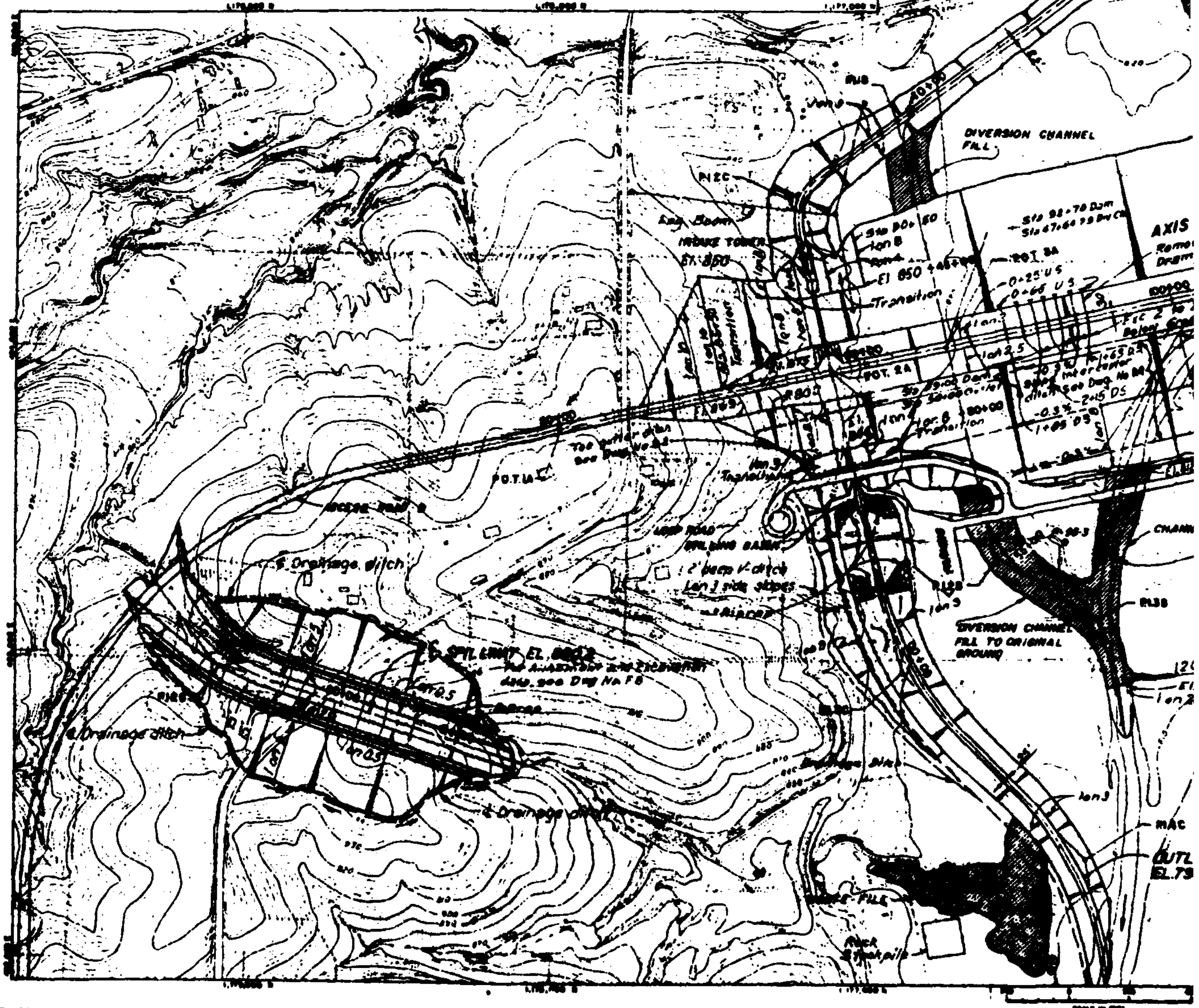
(5) For a viable dam safety program it is essential that in-house drill crew forces be able to respond quickly and effectively to dam emergencies. In order to provide this response it is necessary for in-house drill crew forces to have drilling experience in conditions and of the type that would be encountered in a dam emergency. Although the present conditions at Smithville are not an emergency, the opportunity to install relief wells and a collection system under high piezometric head conditions is an excellent means of further honing and refining the expertise that presently exists in the Kansas City District.

DRAWINGS

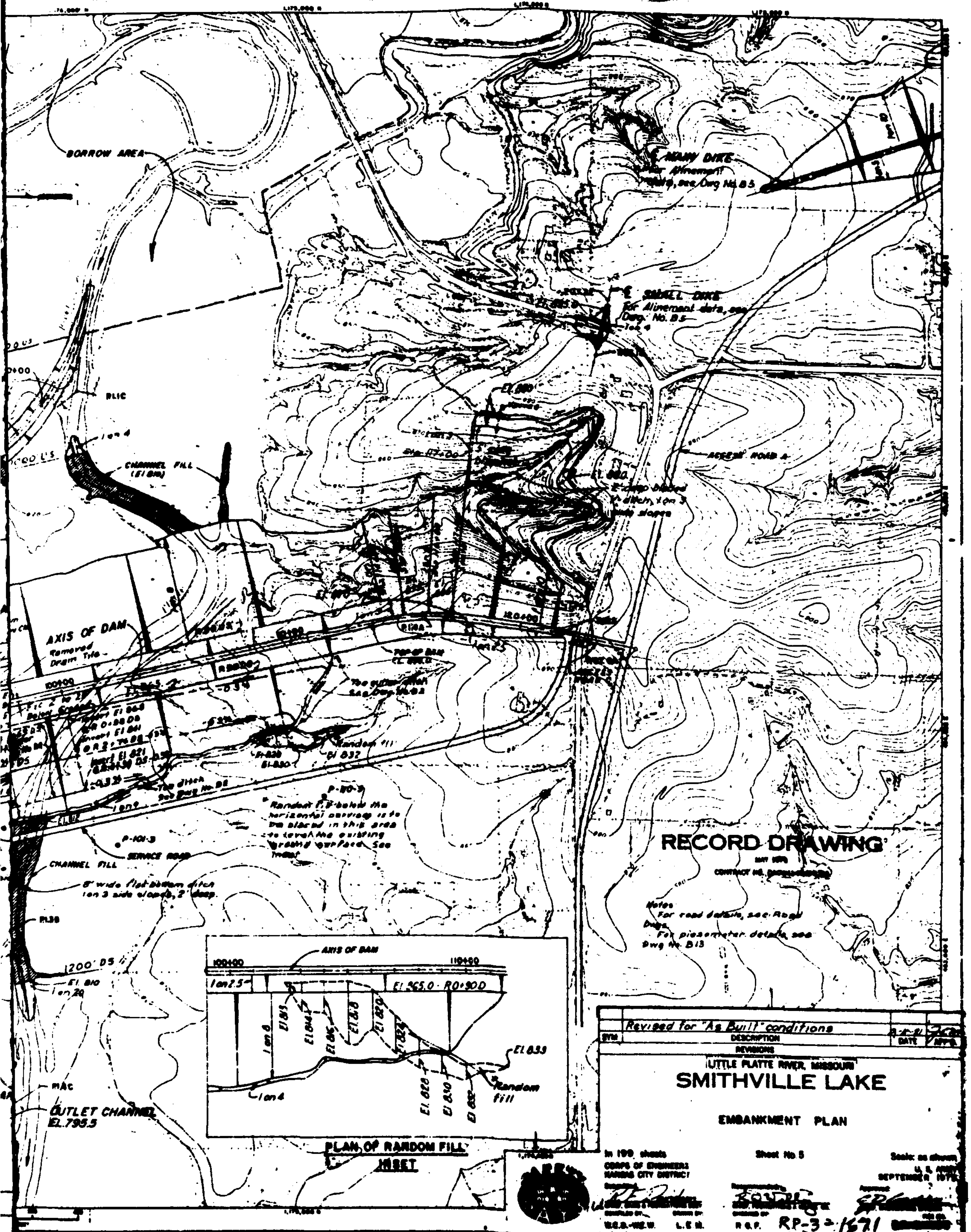
DRAWINGS

ALINEMENT DATA													
P.I.	P.O.T.	LATITUDE	DEPARTURE	AZIMUTH	DISTANCE	STATION	P.C. STA.	P.T. STA.	Δ	D	T	R	L
AXIS DAM													
	1A	1,177,392.67	483,720.20			60+00							
				347°30'	900.00								
	2A	1,177,094.00	483,915.00			89+00 = Station 50+00 Outlet Works							
				347°30'	370.00								
	3A	1,176,732.77	483,995.08			92+70 = Station 47+64.29 Diversion Channel							
				347°30'	2281.63								
4A		1,174,505.22	484,488.92				114+00.09	117+00.09	20°Rt	6°40'	151.54'	859.44'	900.00
				7°30'	640.00								
5A		1,173,870.70	484,405.88				119+88.47	123+88.47	4°Rt	1°	200.08	5729.58	400.00'
				11°30	300.00								
	6A	1,173,576.72	484,345.57			124+88.59							
DIVERSION CHANNEL													
				98°									
1C		1,175,942.23	485,461.96				24+15.48	33+05.48	44°30'Rt	5°	468.81'	1145.92'	890.00
				142°30'	1201.75'								
1B		1,176,896.64	484,729.[illegible]				39+16.75	41+33.42	65°Lt	30°	121.67'	190.98'	216.67'
				77°30'	732.54'								
	3A	1,176,732.77	483,995.08			47+64.29 = Station 92+70 Dam Axis							
				77°30'	700.00'								
2B		1,176,581.26	483,311.67				53+00.18	56+10.18	46°30'Lt	15°	164.11'	381.97'	310.00'
				31°00'	410.00'								
3B		1,176,229.[illegible]	483,100.30				56+91.96	60+01.96	46°30'Rt	15°	164.11'	381.97	310.00'
				77°30'									
OUTLET WORKS													
				98°									
1C		1,175,942.23	485,461.96				24+15.48	33+05.48	44°30'Rt	5°	468.81'	1145.92'	890.00'
				142°30'	1610.00'								
2C		1,177,219.53	484,481.25				43+25.00	45+41.67	65°Lt	30°	121.67'	190.98'	216.67
				77°30'	580.00'								
	2A	1,177,094.00	483,915.00			50+00 = Station 89+00 Dam Axis							
				77°30'	1020.73'								
3C		1,176,873.07	482,918.67				57+50	62+70	32°Lt	6°09'	266.97'	931.05'	520.00'
				43°30'	1010.21'								
4C		1,176,[illegible]	[illegible],197.94				65+34	74+04	51°Rt	6°	455.48'	954.93'	850.00'
				96°30'									





-1-7

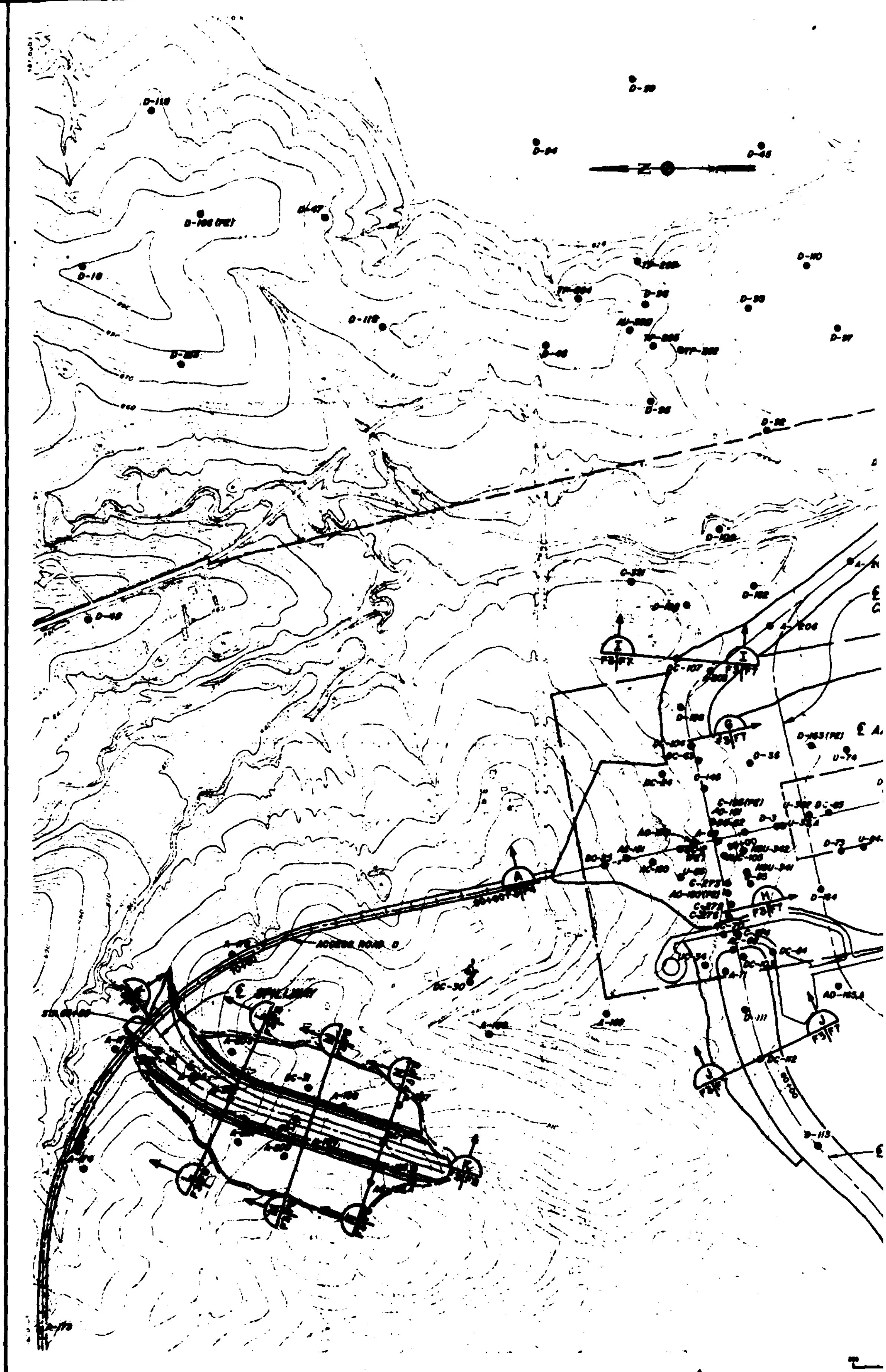


APPENDIX A

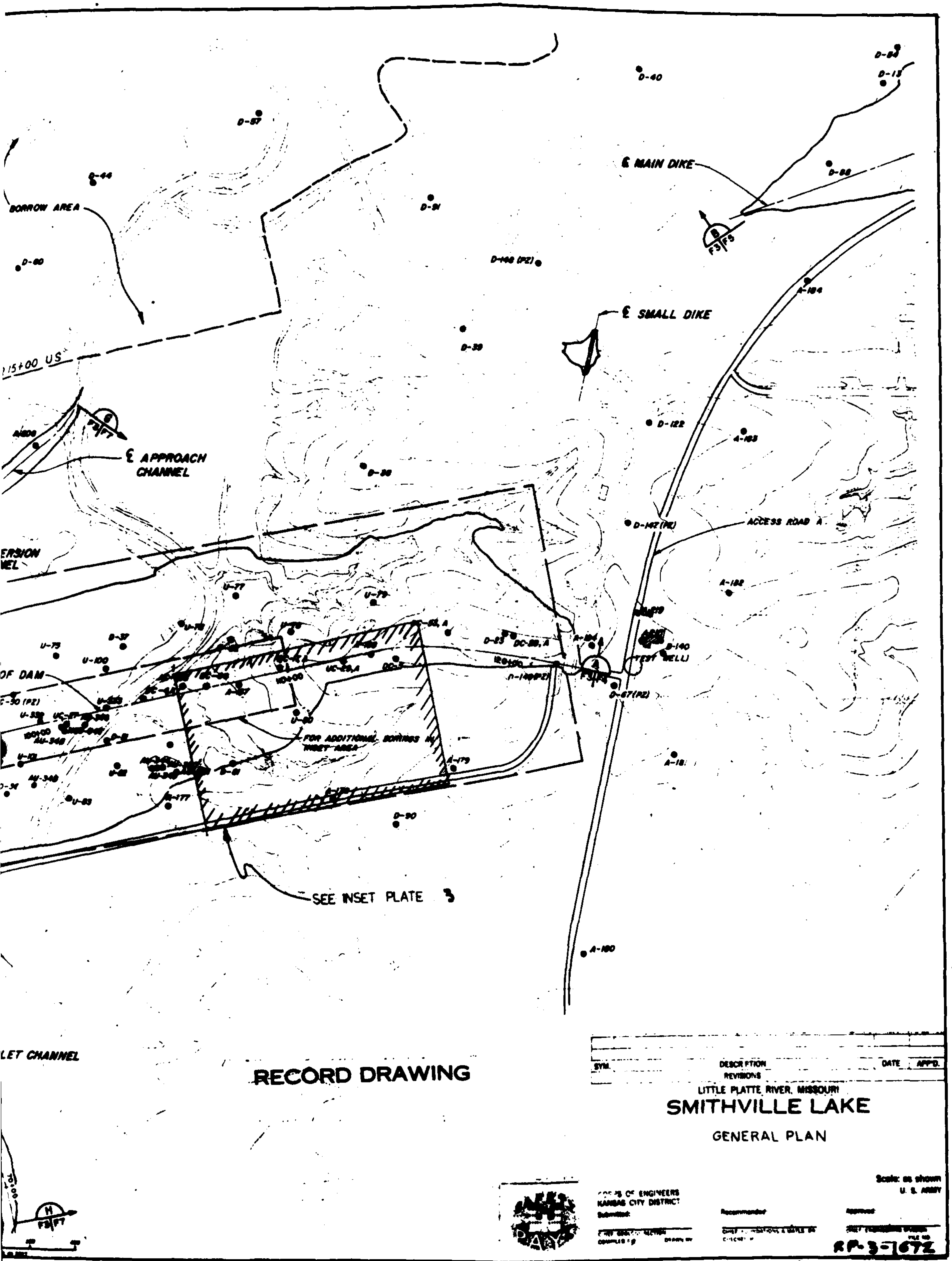
CONSTRUCTION FOUNDATION REPORT PLATE NO. 1

1

2

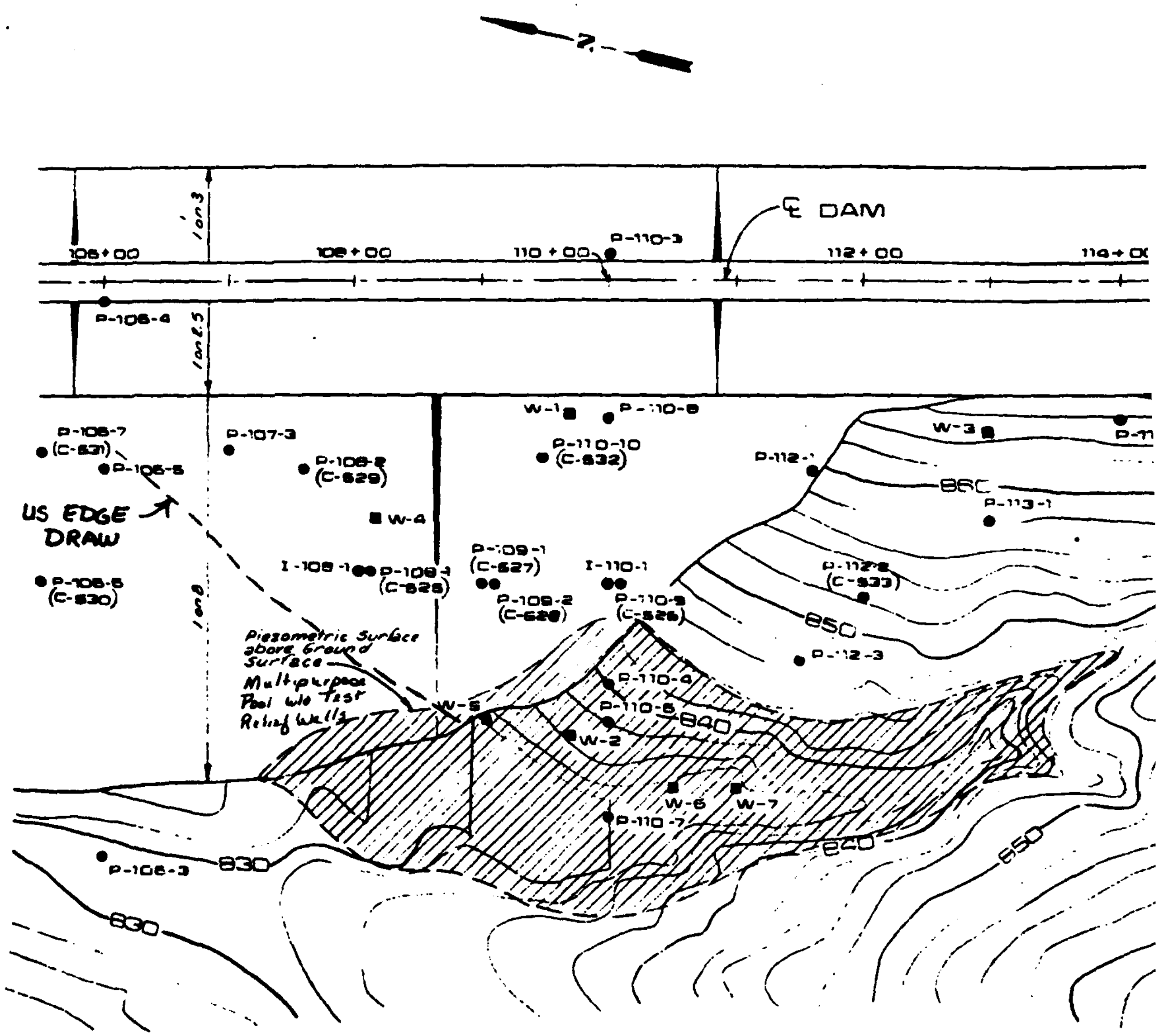


15-1-7



APPENDIX A

PLATE NO 2



PLAN OF EXPLORATIONS
LEFT ABUTMENT
SCALE 1"=50'

-1-7

SYM.	DESCRIPTION	DATE	APPD.
	REVISIONS		

LITTLE PLATTE RIVER, MISSOURI

SMITHVILLE LAKE

PLAN OF EXPLORATIONS - LEFT ABUTMENT

Dwg. No.

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

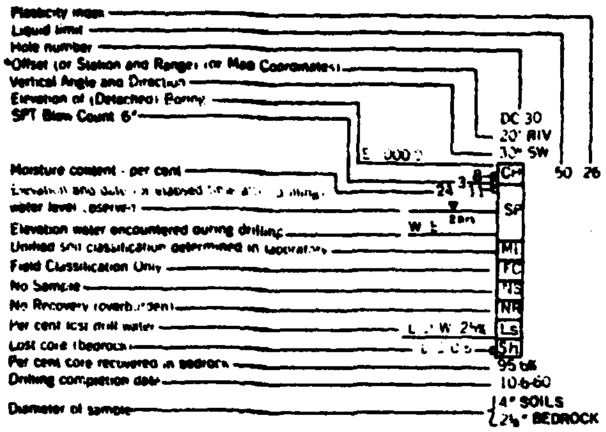
Scale: as shown
U. S. ARMY

Submitted: Recommended: Approved:

RP-3-1673

APPENDIX A PLATE NO. 3

LEGEND FOR LOGS OF BORINGS



*Offset from profile or section may be Upstream or Downstream (U or D), Landward or Riverward (LWD or RIV), or Right or Left (R or L) as defined

MAP SYMBOL

- ● Vertical boring
- 30° Inclined boring showing direction and vertical angle
- ⬤ Calyx boring (30" diameter or larger)

TYPE OF EXPLORATION

CODE DESIGNATION

- D Drive sample hole
- C Core hole
- TP Test pit (includes power auger 24" or larger diameter)
- U Undisturbed sample hole
- A Auger hole hand or power auger less than 24" diameter
- NS Not Sampled (Field Classification from cuttings only)
- FS Field Section of outcrop

BEDROCK UNIT THICKNESS

Parting	< 0.02'
Band	0.02' to 0.2'
Thin Bed	0.2' to 0.5'
Medium bed	0.5' to 1.0'
Thick bed	1.0' to 2.0'
Massive	> 2.0'

UNIFIED SOIL CLASSIFICATION SYSTEM

GW	Well graded gravels, gravel-sand mixtures, little or no fines.
GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
GM	Silty gravels, gravel-sand-silt mixtures.
GC	Clayey gravels, gravel-sand-clay mixtures.
SW	Well-graded sands, gravelly sands, little or no fines
SP	Poorly graded sands or gravelly sands, little or no fines
SM	Silty sands, sand-silt mixtures.
SC	Clayey sands, sand-clay mixtures
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts w/slight plasticity
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
OL	Organic silts and organic silty clays of low plasticity.
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	Inorganic clays of high plasticity, fat clays.
OH	Organic clays of medium to high plasticity, organic silts
Pt	Peat and other highly organic soils

Classification from actual laboratory tests where LL and PI are shown.
Dual classification, where used, is in accordance with the Unified Soil Classification System.
For details on the Unified Soil Classification System, See Waterways Experiment Station Technical Memorandum No. 3-357 dated March 1953 and revised in 1960

TERMS FOR CONSISTENCY OF COHESIVE SOIL AND HARDNESS OF BEDROCK

SOIL

Consistency	Estimated Unconfined Compressive Strength (Tons per square foot)
Very soft	< 0.25
Soft	0.25—0.5
Medium	0.5—1.0
Stiff	1.0—2.0
Very stiff	2.0—4.0
Hard	> 4.0

BEDROCK

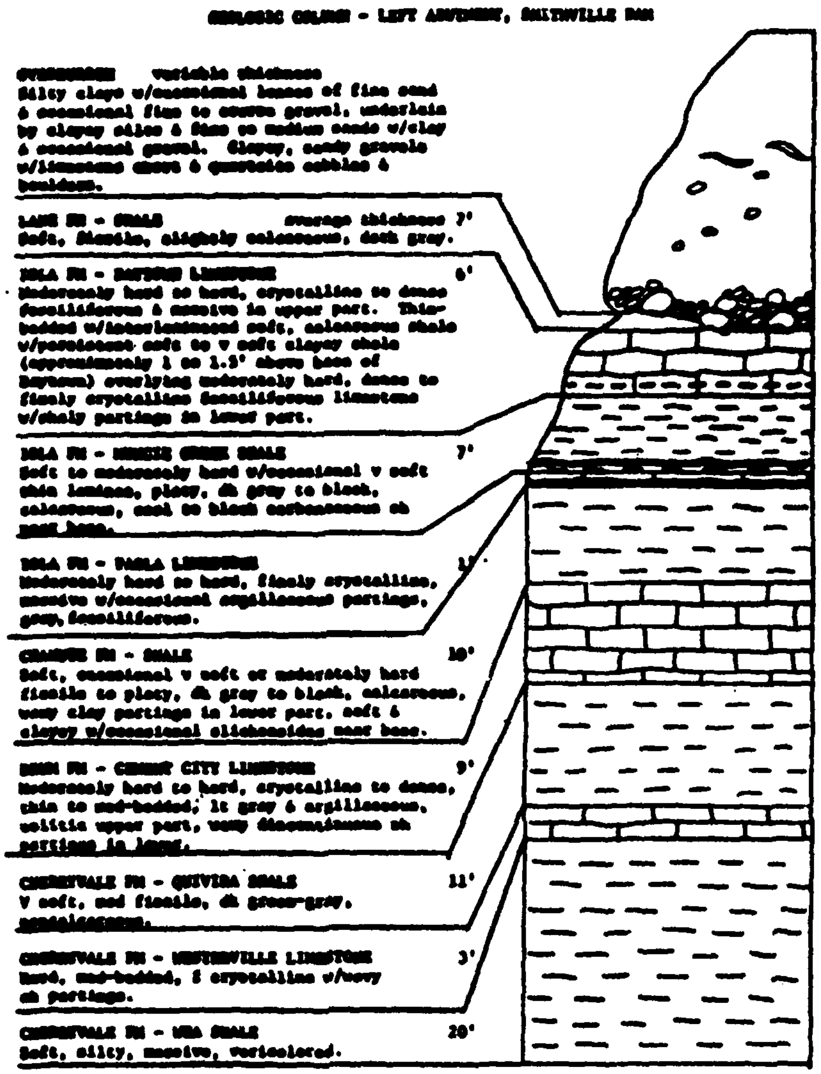
SCALE OF HARDNESS

Very soft or plastic	Can be indented easily with thumb.
Soft	Can be scratched with fingernail.
Moderately hard	Can be scratched easily with knife; cannot be scratched with fingernail.
Hard	Difficult to scratch with knife.
Very Hard	Cannot be scratched with knife.

ABBREVIATIONS

alt	alternating	dmp	damp	lea	leached	rnd (d)	round, (rounded)
ang	angular	dol (c)	dolomite, (dolomitic)	lig	lignite	sat	saturated
an	anhydrite	ext	extremely	ls	limestone	scat	scattered
ar	argillaceous	f (y)	fine, (finely)	lt	light	sd (y)	sand, (sandy)
bdd	bed, bedded, bedding	fe	iron	lo	loose	sev	several
bdr	bedrock	fld	filled	L.C.	lost core	sh (y)	shale, (shaly)
bky	blocky	frm	firm	L.D.W.	lost drill water	si (y)	silt, (silty)
bl	blue	fos (s)	fossil, (fossiliferous)	med	medium	sls	siltstone
bld	boulder	frac (d)	fractures, (fractured)	mic	micaceous	sl	slightly
blk	black	frag (d)	fragments, (fragmented)	min	mineralized	slcs	siliceous
brec (d)	breccia, (brecciated)	fri	friable	mod (y)	moderate, (moderately)	slks	slickensides
brk	broken	fsl	fissile	mot	mottled	so	soft
brn	brown	gr	grain	mss	massive	sol (d)	solution, (solutionized)
c	coarse	gra	gradation	mst	moist	ss	sandstone
calc	calcareous	grn	green	mtl	material	st (g)	stained, (staining)
carb	carbonaceous	grv (y)	gravel, (gravelly)	mtx	matrix	stf	stiff
cav	cavity	gry	gray	nod	nodules	sty	stylolitic
cbl	cobble	gyp	gypsum	num	numerous	v	very
cht	chert	ha	high angle	occ (y)	occasional, (occasionally)	vert	vertical
circl	circulation	hd	hard	op	open	vly	valley
cl (y)	clay (clayey)	hlg	healed	or	orange	w	water
cld	closed	hor	horizontal	org	organic	w/	with
cmtd	cemented	intbd	interbedded	par	partially	wth	weathered
col	columnar	incl	inclusions	pit	pit, pitted, pitting	wht	white
conc	concretions	intlam	interlaminated	pl	plastic	x-bdd	crossed-bedded
cong	conglomerate	irr	irregular	pla	platy	xln	crystalline
crm	crumbly	jt (s)	joint, (joints)	pln	plane	y	yellow
d	dense	la	low angle	ptg (s)	parting, (partings)	When used as log symbol	
dk	dark	lam (d)	laminae, (laminated)	qtz (e)	quartz, (quartzite)	first letter is capitalized	

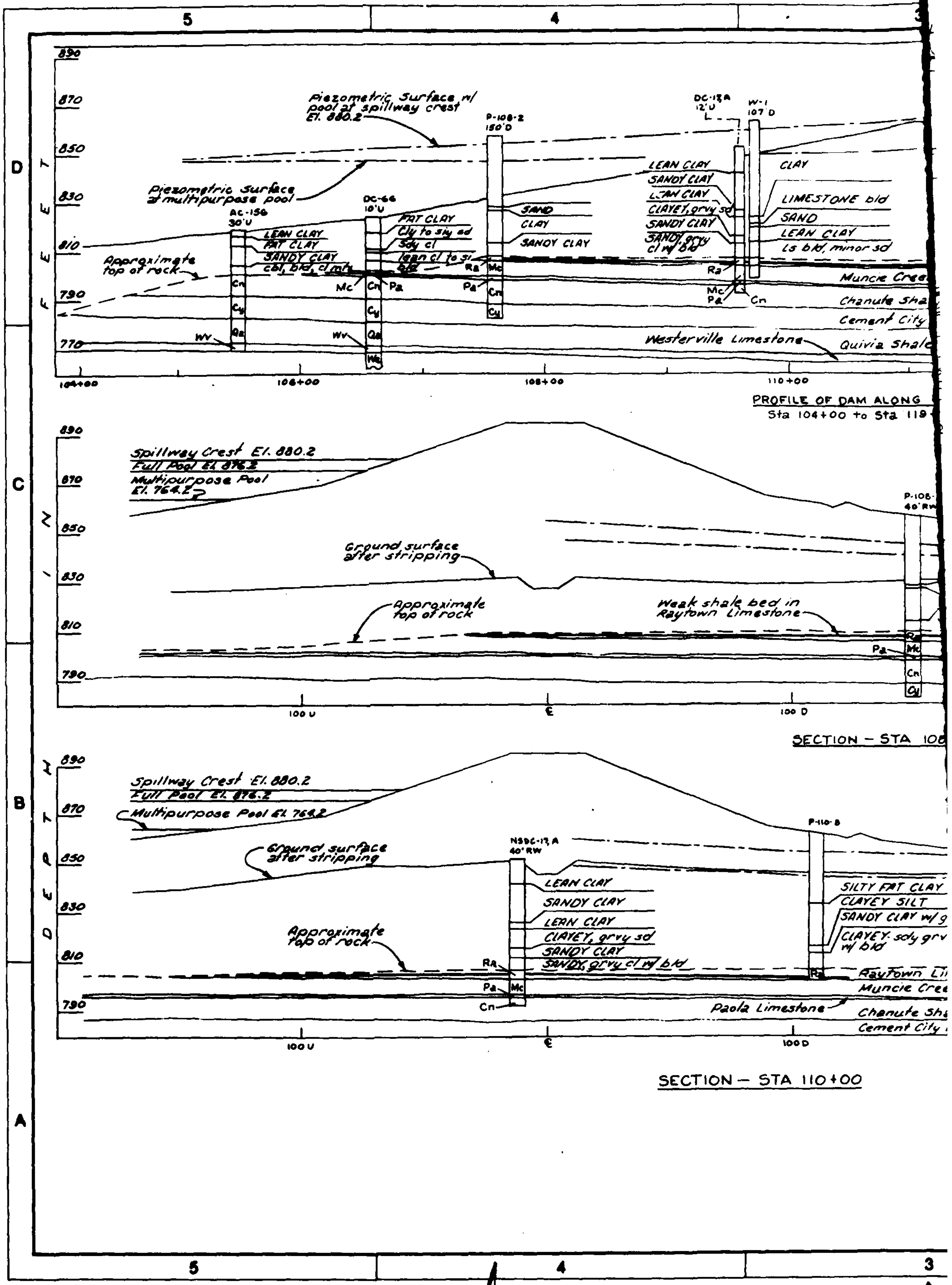
15-1-7



SMITHVILLE DAM
LEFT ABUTMENT

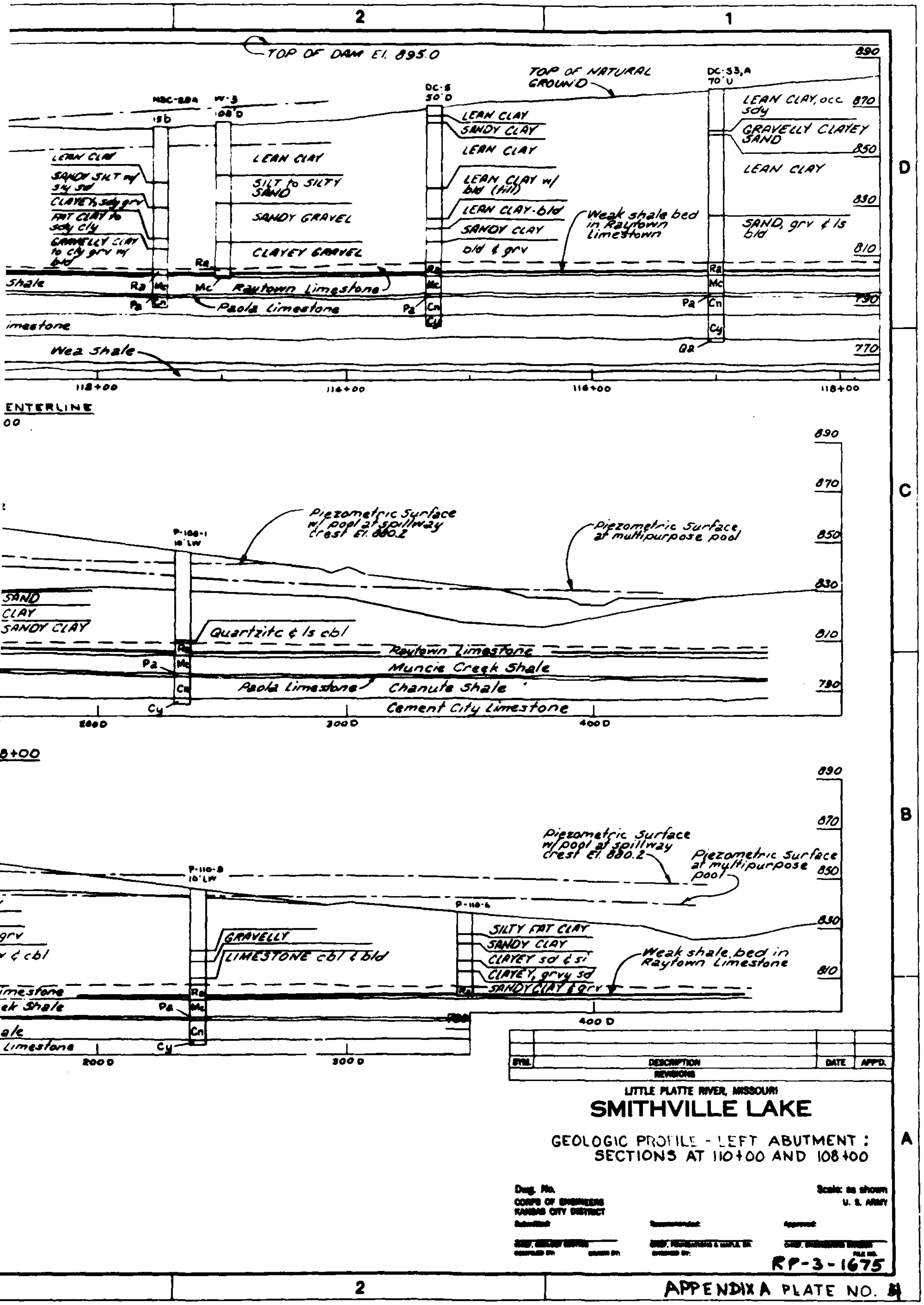
BORING LEGEND
GEOLOGIC Column-
LEFT ABUTMENT

APPENDIX A RP-3-1674
PLATE 3A JULY 1984



Piezometric Surface w/ pool at spillway crest El. 880.2
Piezometric surface at multipurpose pool
Approximate top of rock
AC-156 30'U
DC-66 10'U
P-108-2 150'D
DC-13A 12'U
W-1 107'D
LEAN CLAY
FAT CLAY
SANDY CLAY
cbl, bld, cl mtx
Cly to sly sd
Sdy cl
lean cl to si
bld
SAND
CLAY
CLAYEY, grvy sd
SANDY, grvy cl w/ bld
LIMESTONE bld
Ls bld, minor sd
Muncie Cree
Chanute Sha
Cement City
Westerville Limestone
Quivia Shale
104+00
106+00
108+00
110+00
PROFILE OF DAM ALONG
Sta 104+00 to Sta 119
FEET
890
870
850
830
810
790
770
Spillway Crest El. 880.2
Full Pool El. 876.2
Multipurpose Pool El. 764.2
Ground surface after stripping
Weak shale bed in Raytown Limestone
P-108- 40'RW
100 U
100 D
SECTION - STA 108
DEPTH IN
NSDC-13A 40'RW
P-110-8
SILTY FAT CLAY
CLAYEY SILT
SANDY CLAY w/ g
CLAYEY, sdy grv w/ bld
Raytown Li
Muncie Cree
Paola Limestone
Chanute Sh
Cement City
SECTION - STA 110+00

-1-7



TOP OF DAM El. 895.0
TOP OF NATURAL GROUND
DC-53,A 70'U
DC-5 50'D
W-3 108'D
LEAN CLAY
SANDY SILT w/ sty sd
CLAYEY sdy grv
FAT CLAY to sdy cly
GRAVELLY CLAY to cly grv w/ bld
SILT to SILTY SAND
SANDY GRAVEL
CLAYEY GRAVEL
SANDY CLAY
LEAN CLAY w/ bld (fill)
LEAN CLAY-bld
bld & grv
LEAN CLAY, occ sdy
GRAVELLY CLAYEY SAND
SAND, grv & ls bld
Weak shale bed in Raytown Limestone
Shale
Raytown Limestone
Paola Limestone
Wea Shale
ENTERLINE
Piezometric Surface w/ pool at spillway crest El. 880.2
Piezometric Surface at multipurpose pool
P-108-1 10'LW
SAND
CLAY
SANDY CLAY
Quartzite & ls cbl
Raytown Limestone
Muncie Creek Shale
Paola Limestone
Chanute Shale
Cement City Limestone
8+00
P-110-8 10'LW
GRAVELLY
LIMESTONE cbl & bld
SILTY FAT CLAY
SANDY CLAY
CLAYEY sd & si
CLAYEY, grvy sd
SANDY CLAY & grv
Weak shale bed in Raytown Limestone
Limestone
Creek Shale
ale
Limestone
890
870
850
830
810
790
770
112+00
114+00
116+00
118+00
2000
3000
400 D
SYM.
DESCRIPTION
DATE
APPD.
REVISIONS
LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
GEOLOGIC PROFILE - LEFT ABUTMENT : SECTIONS AT 110+00 AND 108+00
Dwg. No.
CORPS OF ENGINEERS KANSAS CITY DISTRICT
Scale: as shown
U. S. ARMY
RP-3-1675
APPENDIX A PLATE NO. 8

2

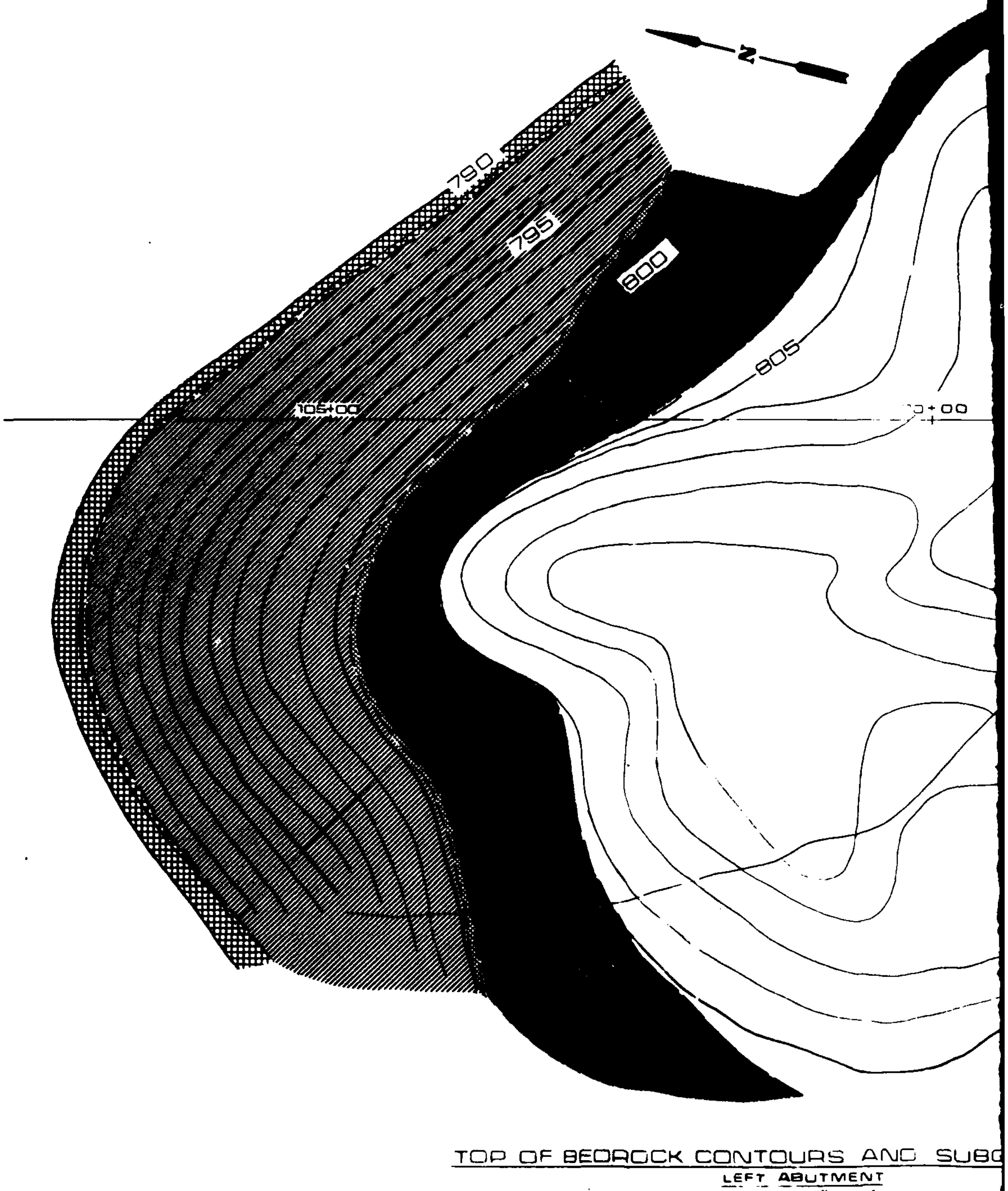
5	4	

D

C

B

A

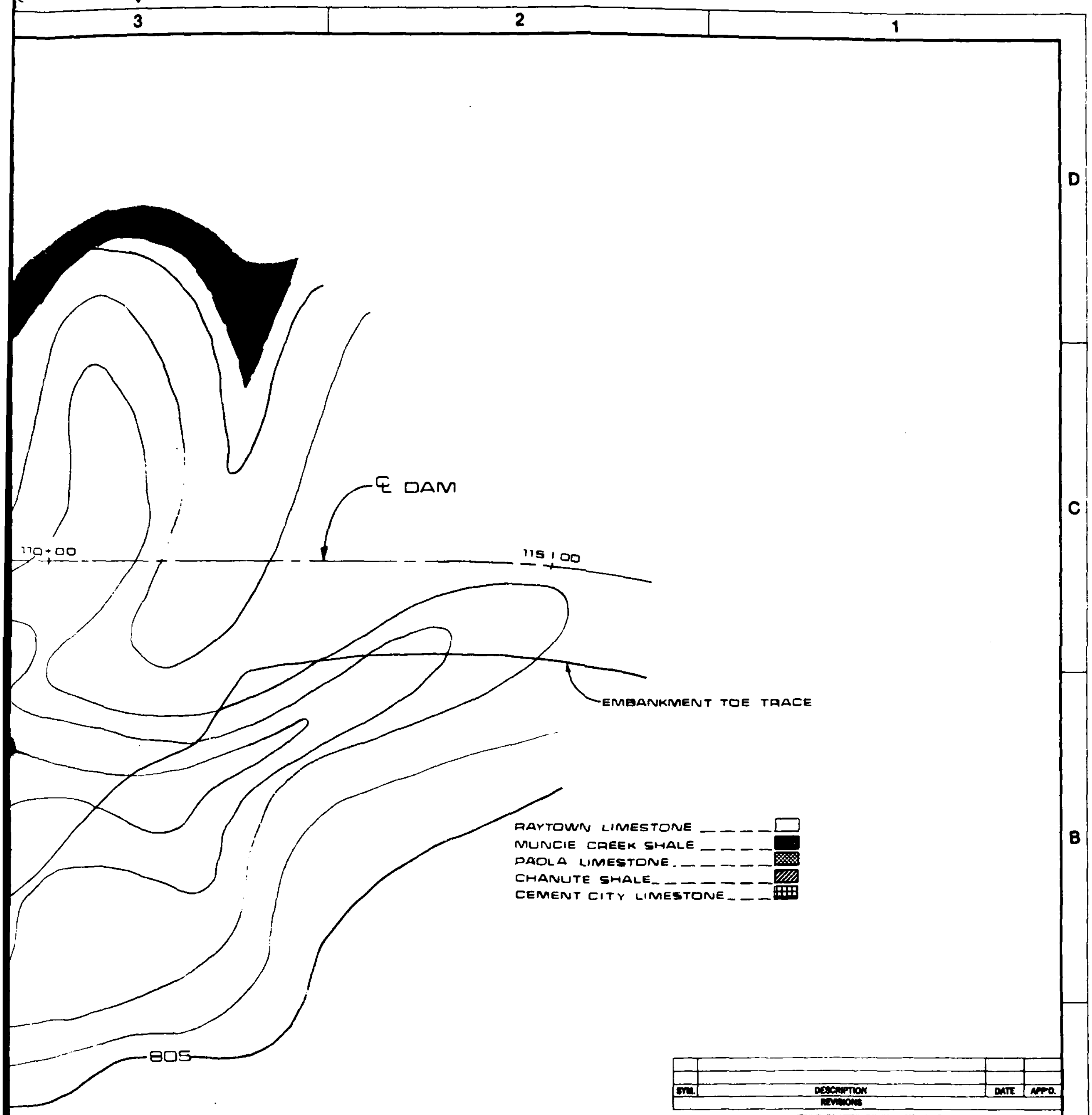


TOP OF BEDROCK CONTOURS AND SUBC

LEFT ABUTMENT
SCALE 1" = 50'

5	4	

15-1-7



SYM.	DESCRIPTION	DATE	APP'D.

REVISIONS

LITTLE PLATTE RIVER, MISSOURI

SMITHVILLE LAKE

TOP OF BEDROCK CONTOURS
AND SUBCROP MAP
LEFT ABUTMENT

Dwg. No.

Scale: as shown

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

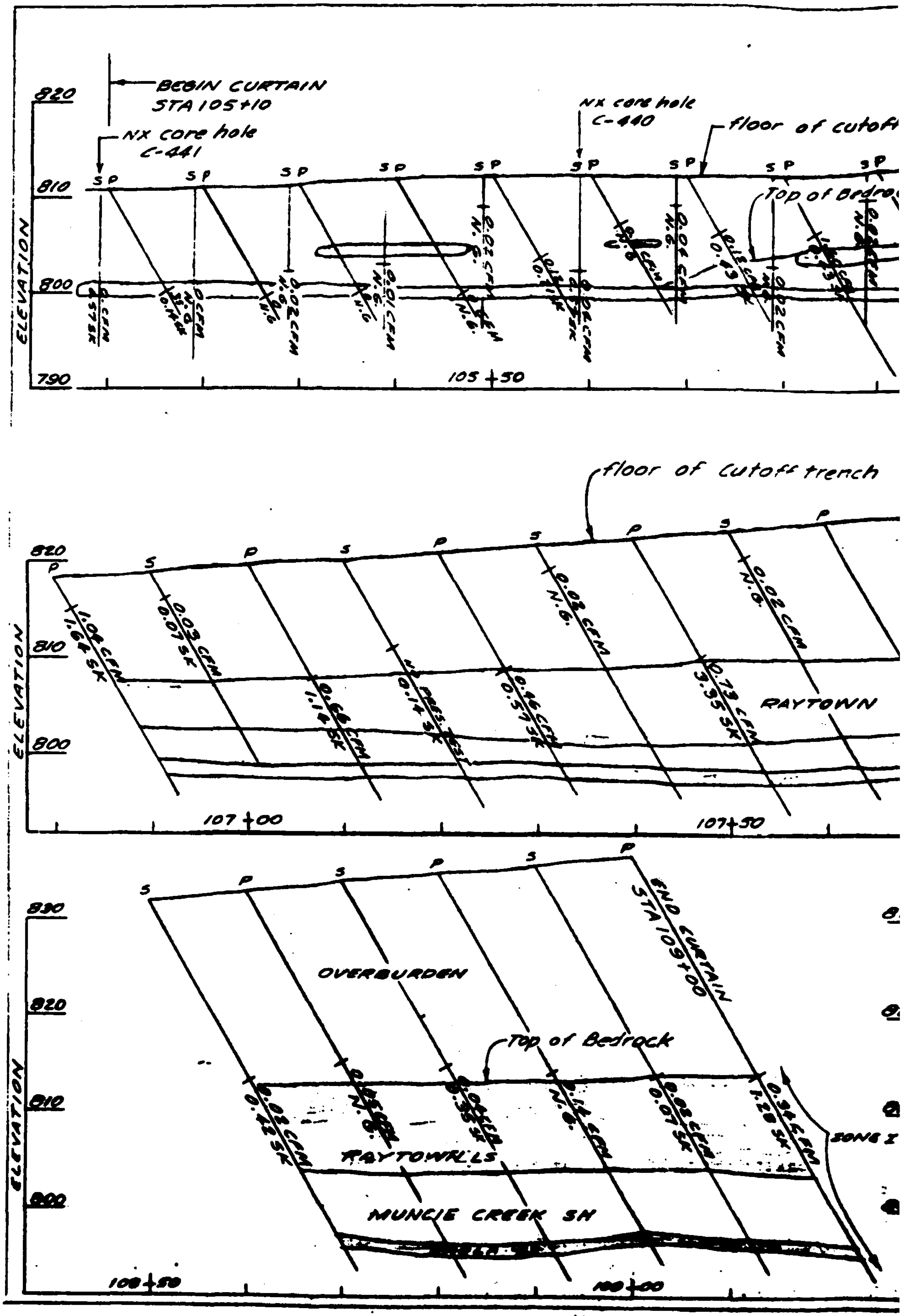
U. S. ARMY

Submitted: Recommended: Approved:

COMPILED BY: DRAWN BY: CHIEF, CHECKED BY: CHIEF, ENGINEERING DIVISION

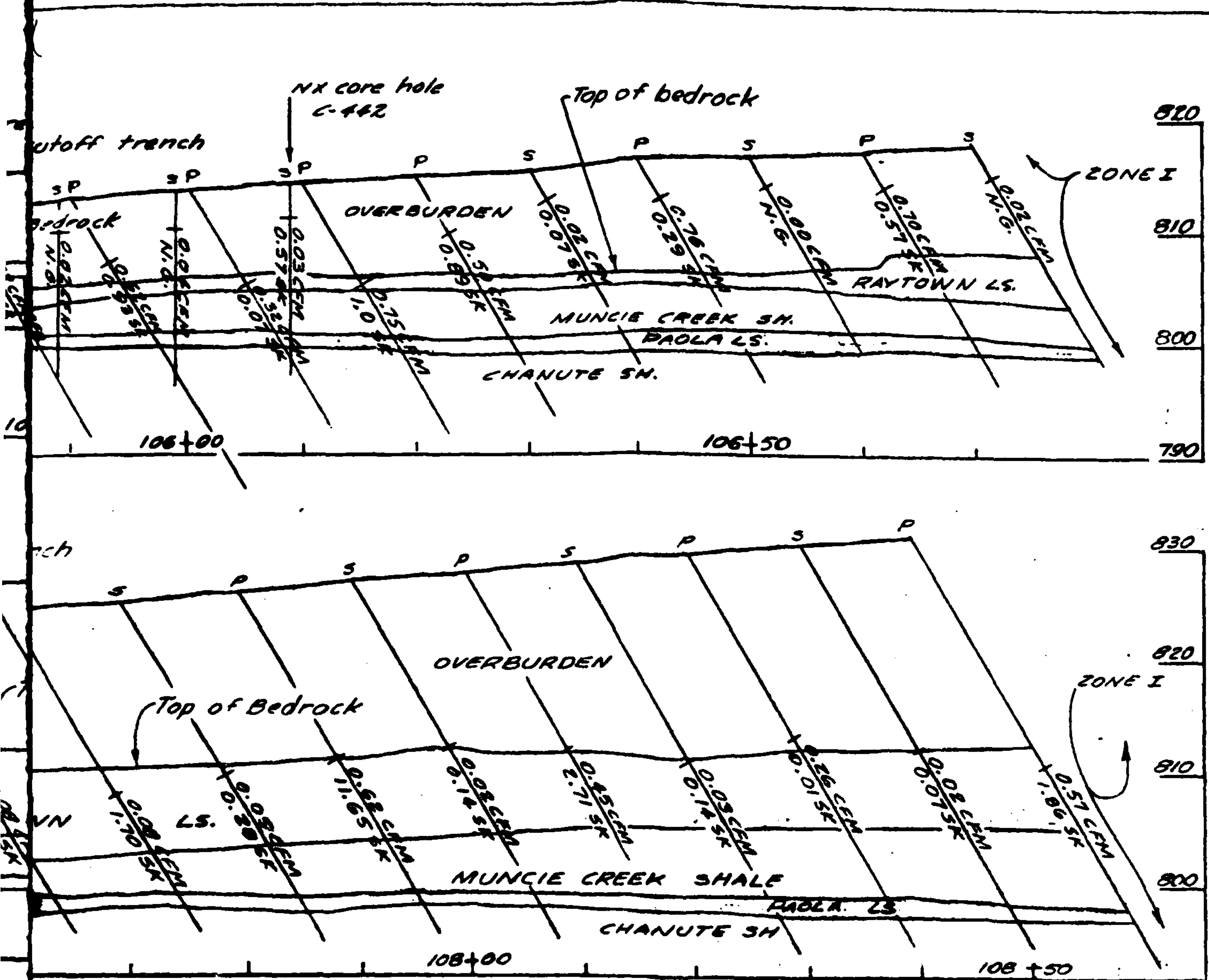
FILE NO. RP-3-1676

APPENDIX A PLATE NO. 5


BEGIN CURTAIN
STA 105+10
NX core hole
C-441
NX core hole
C-440
floor of cutoff
Top of Bedrock
ELEVATION
820
810
800
790
105+50
floor of cutoff trench
RAYTOWN
107+00
107+50
830
OVERBURDEN
END CURTAIN
STA 109+00
Top of Bedrock
RAYTOWN LS
MUNCIE CREEK SH
ZONE I

5-1-7

1



LEFT ABUTMENT GROUT CURTAIN PROFILE

STA 105+10 TO STA 108+95

LOOKING UPSTREAM

LEGEND

P Primary hole
S Secondary hole
N.G. Hole not pressure grouted

Packer depth
Pressure Test: Water take in gallons per minute
Grout take in sacks of cement

Grout holes inclined 30° Landward and parallel to dam axis

0 10
SCALE IN FEET

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE

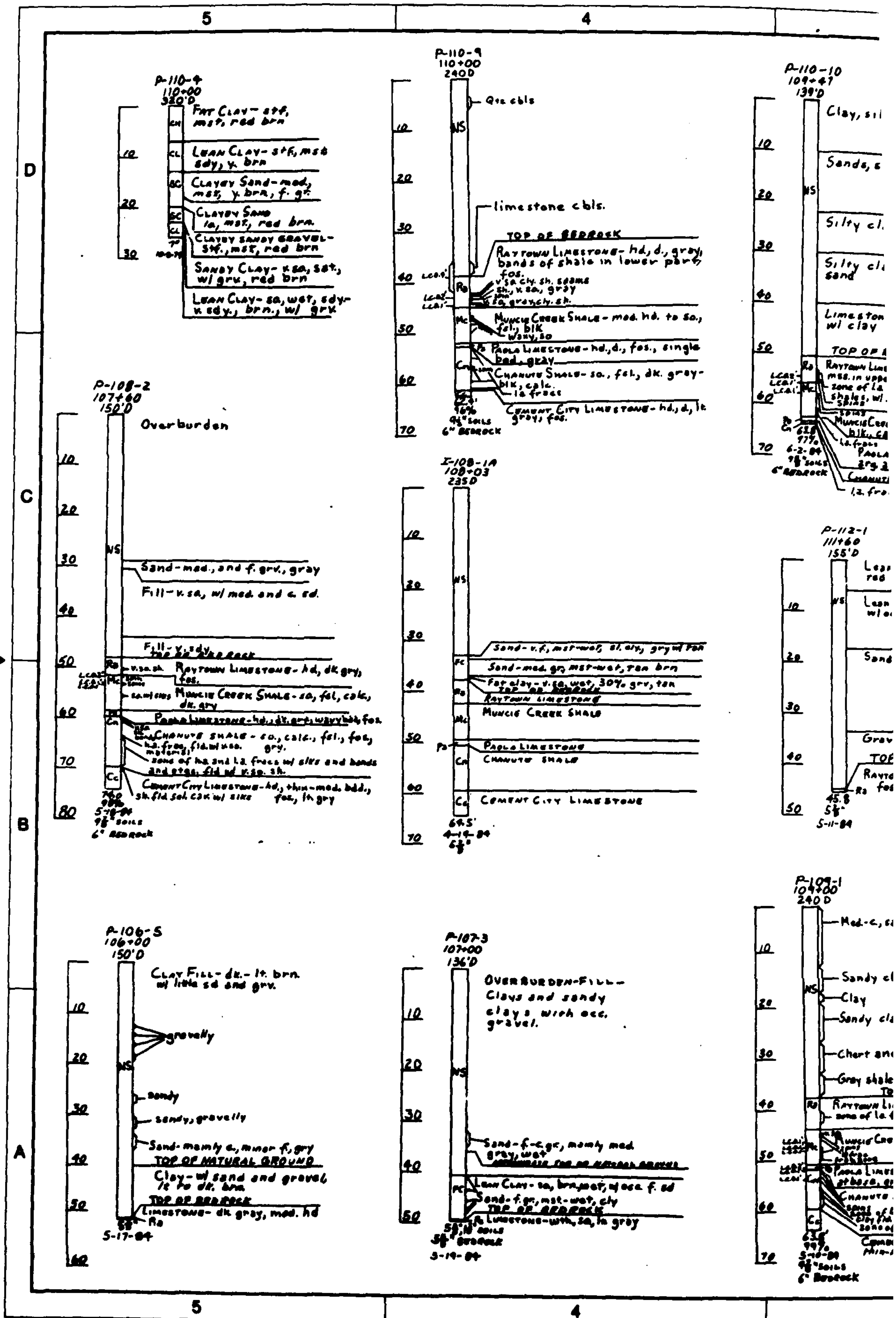
LEFT ABUTMENT GROUT CURTAIN PROFILE

In 1 sheet — Sheet No. 1 — Scale: as shown

CORPS OF ENGINEERS U. S. ARMY
KANSAS CITY DISTRICT

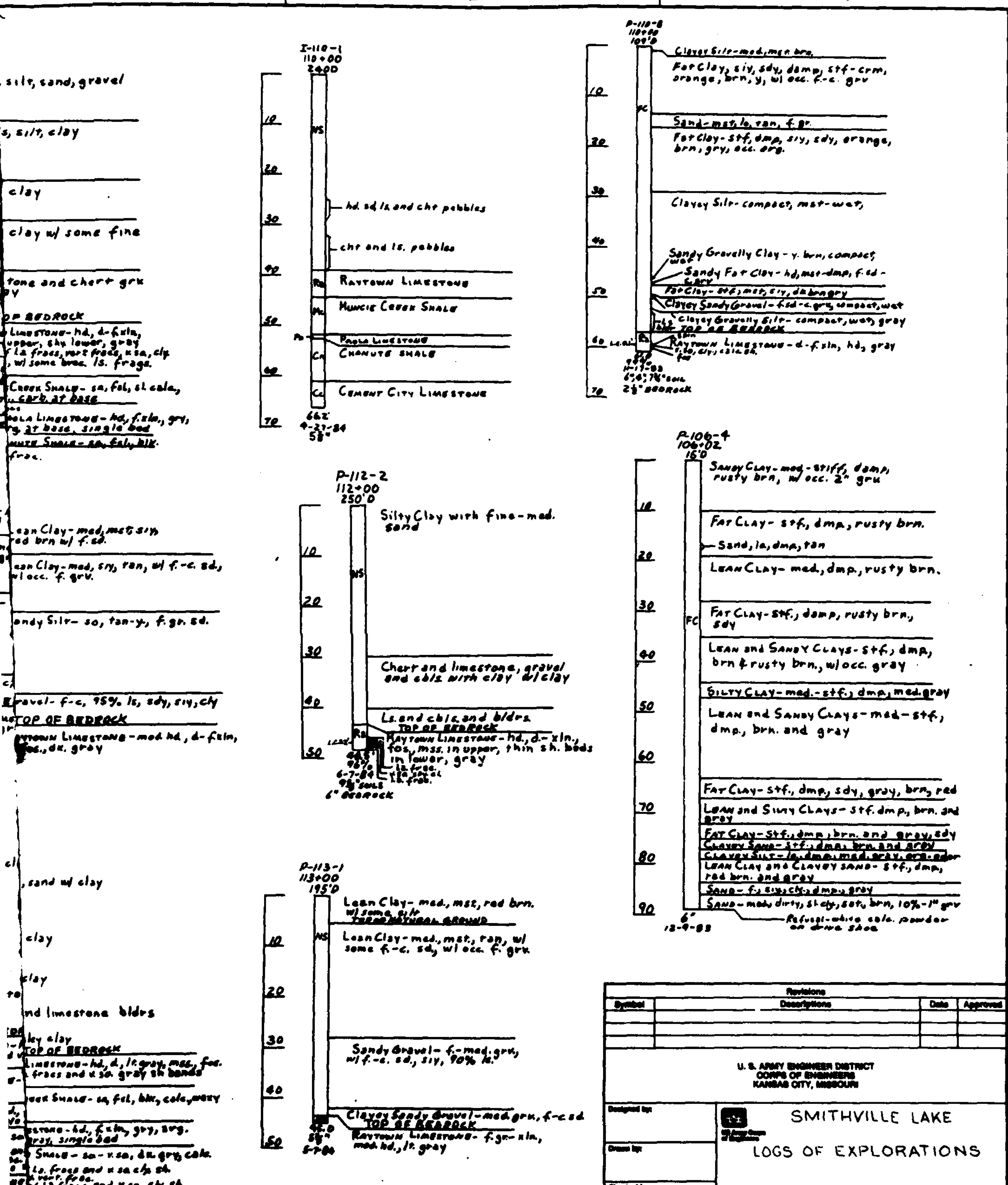
FILE NO. RP-3-[illegible] 1677

AUGUST 1981



5
4
D
C
B
A
P-110-4
110+00
330'D
Fat Clay- stf, mst, red brn
Lean Clay- stf, mst, sdy, y. brn
Clayey Sand- med, mst, y. brn, f. gr.
Clayey Sand lo, mst, red brn
Clayey sandy gravel- stf., mst, red brn
Sandy Clay- v.so, sat, w/ grv, red brn
Lean Clay- so, wet, sdy- v.sdy., brn., w/ grv.
P-110-9
110+00
240D
Qtz cbls
limestone cbls.
TOP OF BEDROCK
Raytown Limestone- hd, d., gray, bands of shale in lower part, fos.
Muncie Creek Shale- med. hd. to so., fsl, blk
Paola Limestone- hd., d., fos., single bed, gray
Chanute Shale- so., fsl, dk. grey-blk, calc.
Cement City Limestone- hd, d, lt gray, fos.
6" Bedrock
P-110-10
109+47
139'D
Clay, sil
Sands, s
Silty cl
Silty cl sand
Limeston w/ clay
P-108-2
107+60
150'D
Overburden
Sand- med., and f. grv., gray
Fill- v.so, w/ med. and c. sd.
Raytown Limestone- hd, dk gry, fos.
Muncie Creek Shale- so, fsl, calc, dk. gry
Paola Limestone- hd., dk. grey, wavy bdd., fos.
Chanute shale- so, calc., fsl., fos, gry.
Cement City Limestone- hd, thin-med. bdd., fos, lt gry
9 1/2" soils
6" Bedrock
I-108-1A
108+03
235D
Sand- med. gr, mst-wet, tan brn
Raytown Limestone
Muncie Creek Shale
Paola Limestone
Chanute Shale
Cement City Limestone
P-112-1
111+60
155'D
5-11-84
P-106-5
106+00
150'D
Clay Fill- dk.- lt. brn. w/ little sd and grv.
gravelly
sandy
sandy, gravelly
Sand- mostly c., minor f, gry
TOP OF NATURAL GROUND
Clay- w/ sand and gravel, lt to dk. brn
TOP OF BEDROCK
Limestone- dk gray, mod. hd
5-17-84
P-107-3
107+00
136'D
Overburden-Fill- Clays and sandy clays with occ. gravel.
TOP OF BEDROCK
5-19-84
P-109-1
109+00
240 D
Sandy cl
Clay
Sandy cl
Chert an
Gray shale
Raytown Li
6" Bedrock

5-1-7



Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: | Drawn by: | Checked by: | Submitted by:

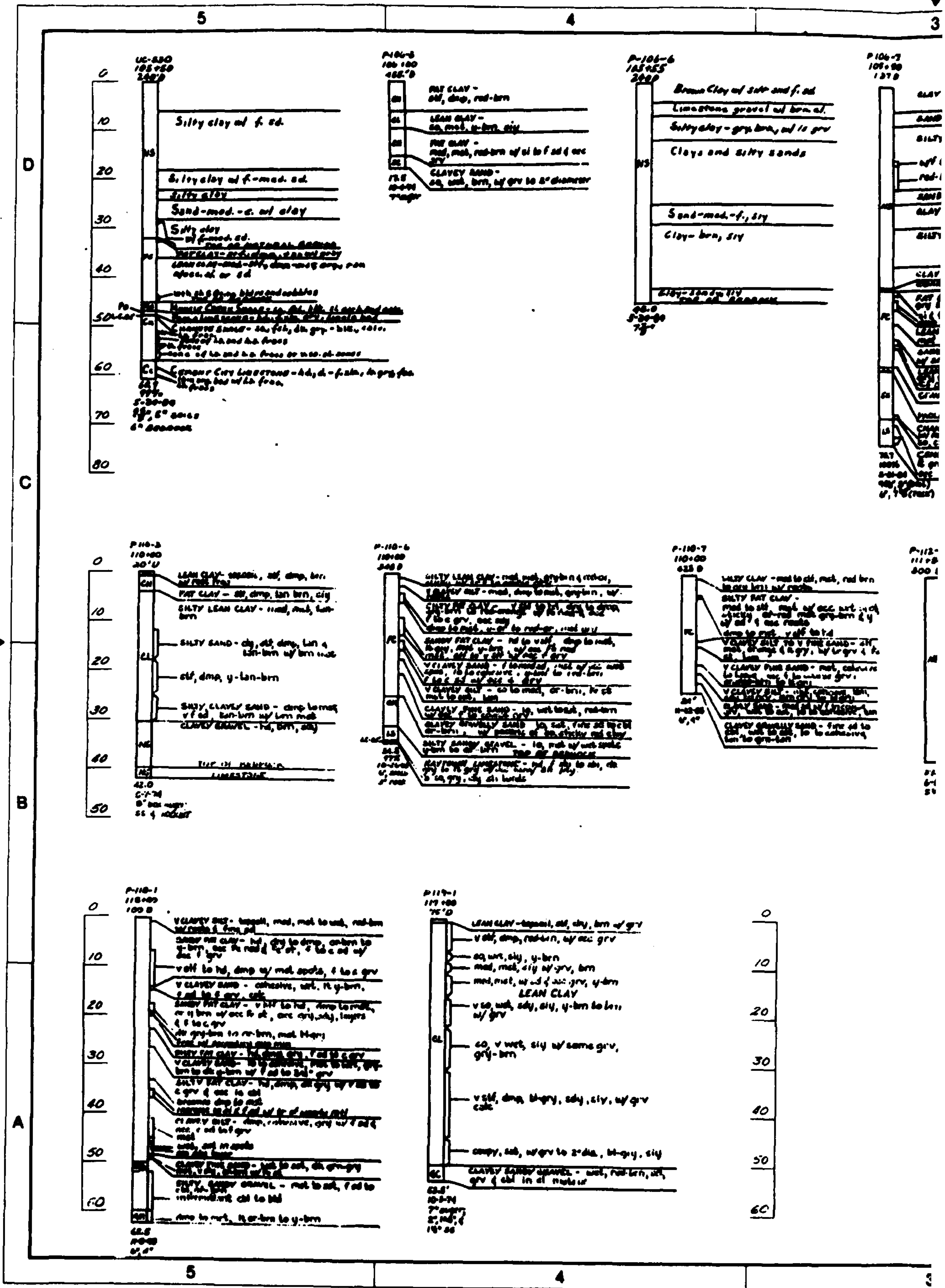
SMITHVILLE LAKE

LOGS OF EXPLORATIONS

Scale: | Sheet number: | Date: | Dwg. No.:

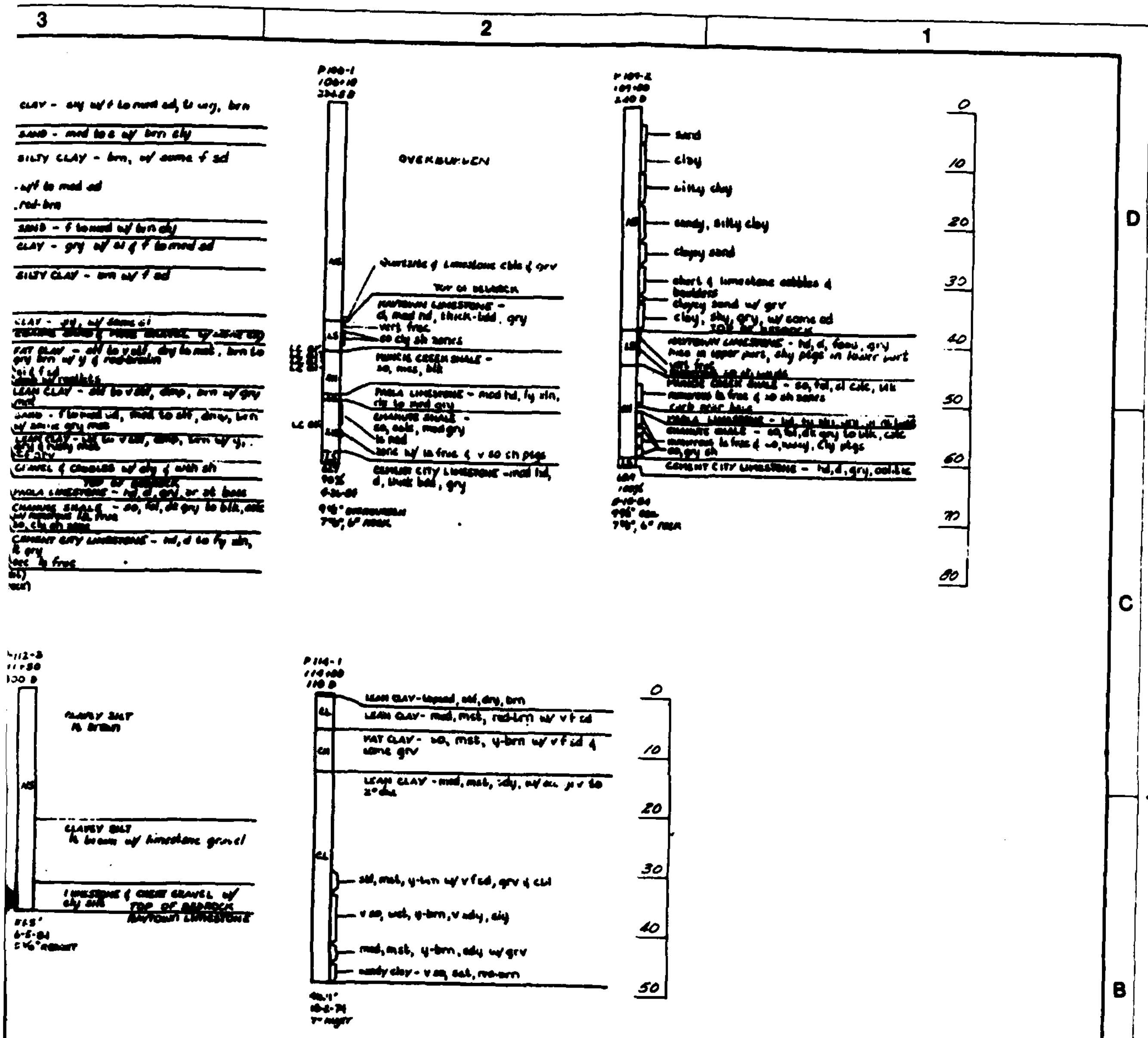
RP-3-1678

APPENDIX A PLATE NO. 9



5
4
3
D
C
B
A
UC-530
P-106-5
P-106-6
P-106-7
P-110-3
P-110-6
P-110-7
P-110-1
P-117-1
0
10
20
30
40
50
60
70
80
Silty clay w/ f. sd.
Silty clay w/ f-med. sd.
Silty clay
Sand-med.-c. w/ clay
FAT CLAY - stf, dmp, red-brn
LEAN CLAY -
CLAYEY SAND -
Brown Clay w/ silt and f. sd.
Limestone gravel w/ brn. cl.
Silty clay - gry. brn, w/ lt grv
Clays and silty sands
Sand-med.-f., sly
Clay- brn, sly
FAT CLAY - stf, dmp, tan brn, sly
SILTY LEAN CLAY - mod, mst, tan-brn
SILTY SAND - sly, stf, dmp, tan & tan-brn w/ brn mot
stf, dmp, y-tan-brn
CLAYEY GRAVEL - hd, brn, dry
LIMESTONE
LEAN CLAY - topsoil, stf, dry, brn w/ grv
v stf, dmp, red-brn, w/ occ grv
co, wet, sly, y-brn
mod, mst, sly w/ grv, brn
LEAN CLAY
co, v wet, sly w/ same grv, gry-brn
v stf, dmp, bl-gry, sdy, sly, w/ grv calc
CLAYEY SANDY GRAVEL

C
-1-7



SYM.	DESCRIPTION	DATE	APP'D.
	REVISIONS		

LITTLE PLATTE RIVER, MISSOURI

SMITHVILLE LAKE

LOGS OF EXPLORATIONS

Scale: as shown

U. S. ARMY

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

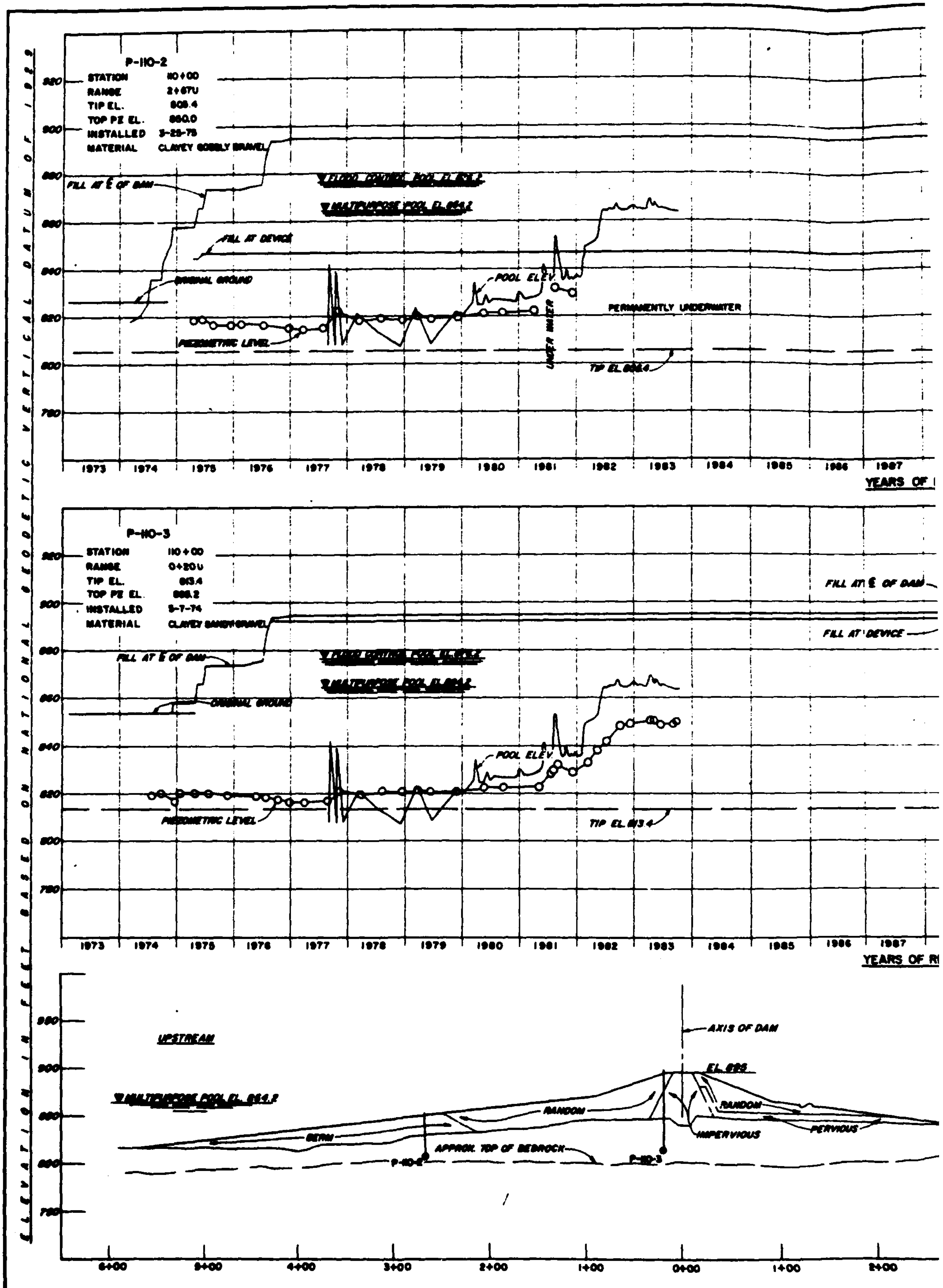
Submitted:

Recommended:

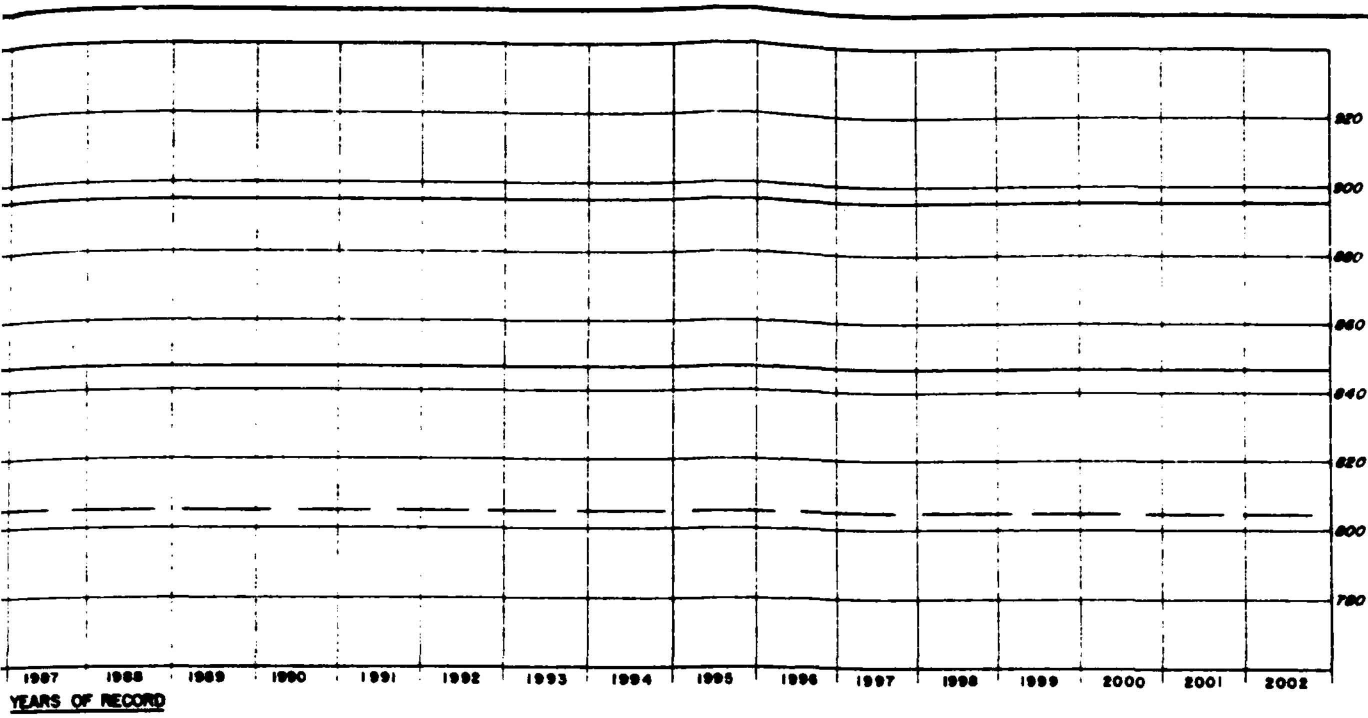
Approved:

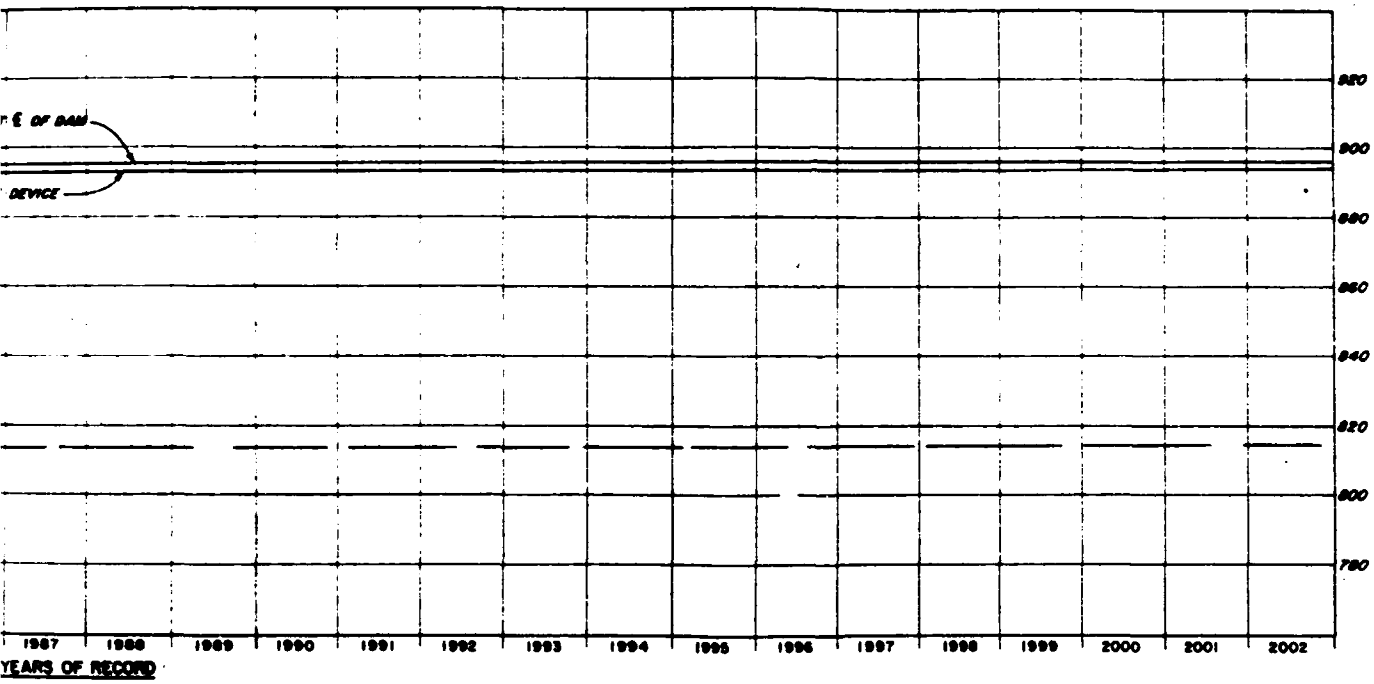
RP-3-16

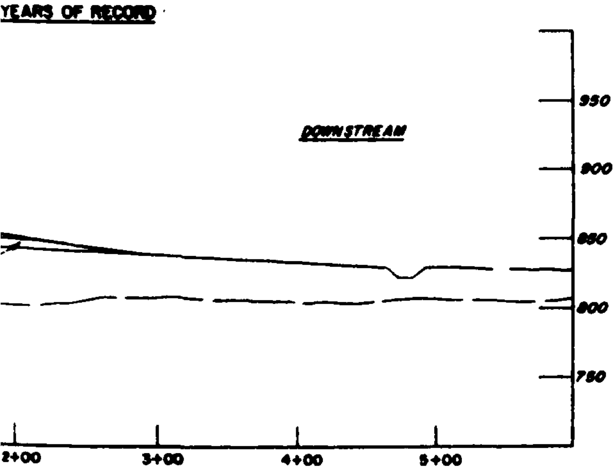
APPENDIX A PLATE NO.


ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929
P-110-2
STATION 110+00
RANGE 2+67U
TIP EL. 808.4
TOP PZ EL. 860.0
INSTALLED 3-25-75
MATERIAL CLAYEY COBBLY GRAVEL
FILL AT ℄ OF DAM
FLOOD CONTROL POOL EL 874.2
MULTIPURPOSE POOL EL 864.2
FILL AT DEVICE
ORIGINAL GROUND
POOL ELEV.
PERMANENTLY UNDERWATER
UNDER WATER
PIEZOMETRIC LEVEL
TIP EL 808.4
1973
1974
1975
1976
1977
1978
1979
1980
1981
1982
1983
1984
1985
1986
1987
YEARS OF
P-110-3
STATION 110+00
RANGE 0+20U
TIP EL. 813.4
TOP PZ EL. 895.2
INSTALLED 5-7-74
MATERIAL CLAYEY SANDY GRAVEL
FILL AT ℄ OF DAM
FILL AT DEVICE
FLOOD CONTROL POOL EL 874.2
MULTIPURPOSE POOL EL 864.2
ORIGINAL GROUND
POOL ELEV.
PIEZOMETRIC LEVEL
TIP EL 813.4
YEARS OF R
UPSTREAM
AXIS OF DAM
EL. 895
MULTIPURPOSE POOL EL 864.2
RANDOM
RANDOM
IMPERVIOUS
PERVIOUS
BERM
APPROX. TOP OF BEDROCK
P-110-2
P-110-3
5+00
4+00
3+00
2+00
1+00
0+00
1+00
2+00

15-1-7







Revised September 1982

LITTLE PLATTE RIVER, MISSOURI

SMITHVILLE LAKE

PERIODIC INSPECTION

PIEZOMETRIC LEVELS
P-110-2 (OPEN TUBE)
P-110-3 (OPEN TUBE)

In 1 sheet | Sheet No. 1 | Scale: as shown

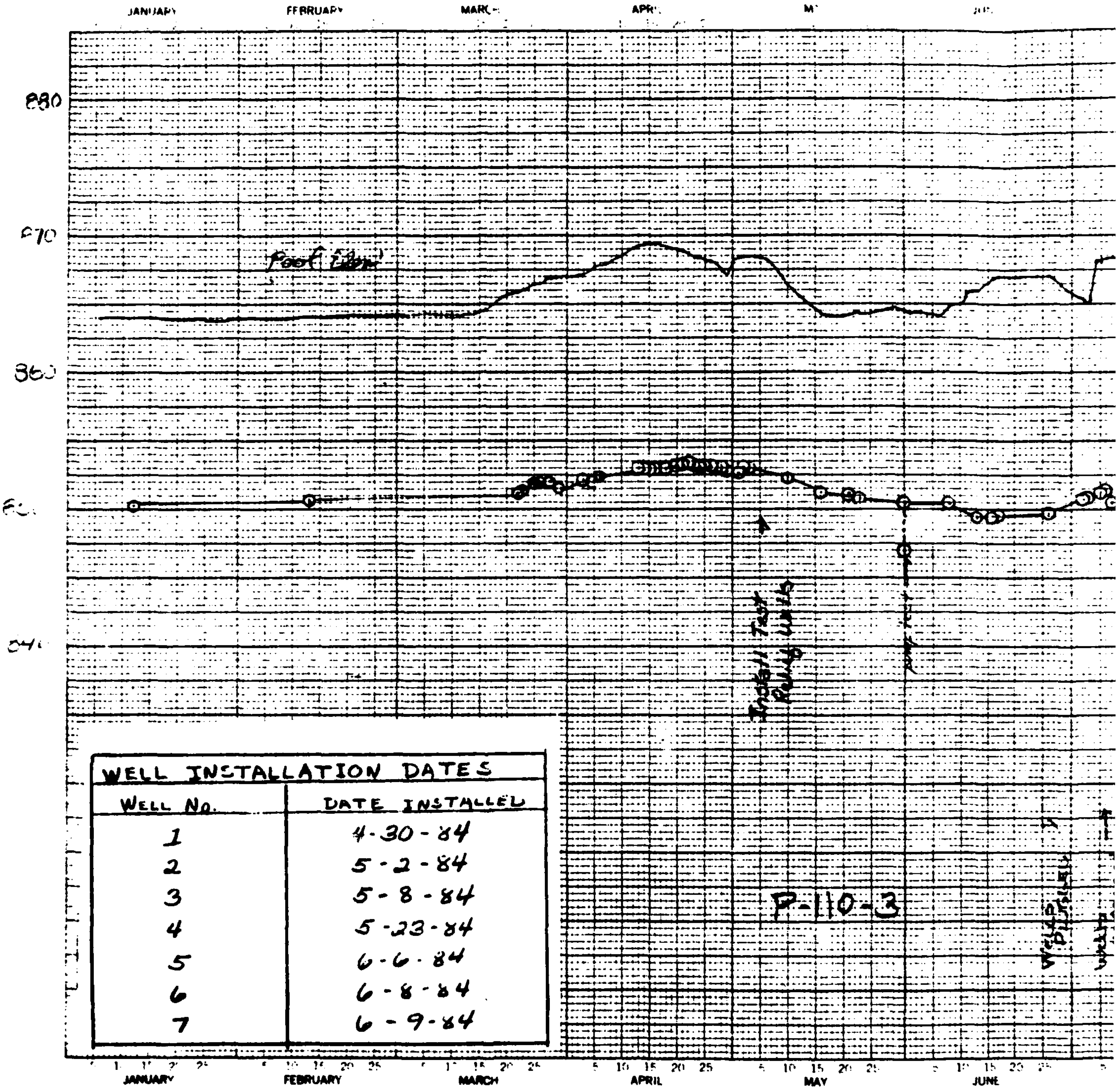
CORPS OF ENGINEERS | U. S. ARMY

KANSAS CITY DISTRICT

FILE NO. RP-3-[redacted] 1690

MARCH 1981

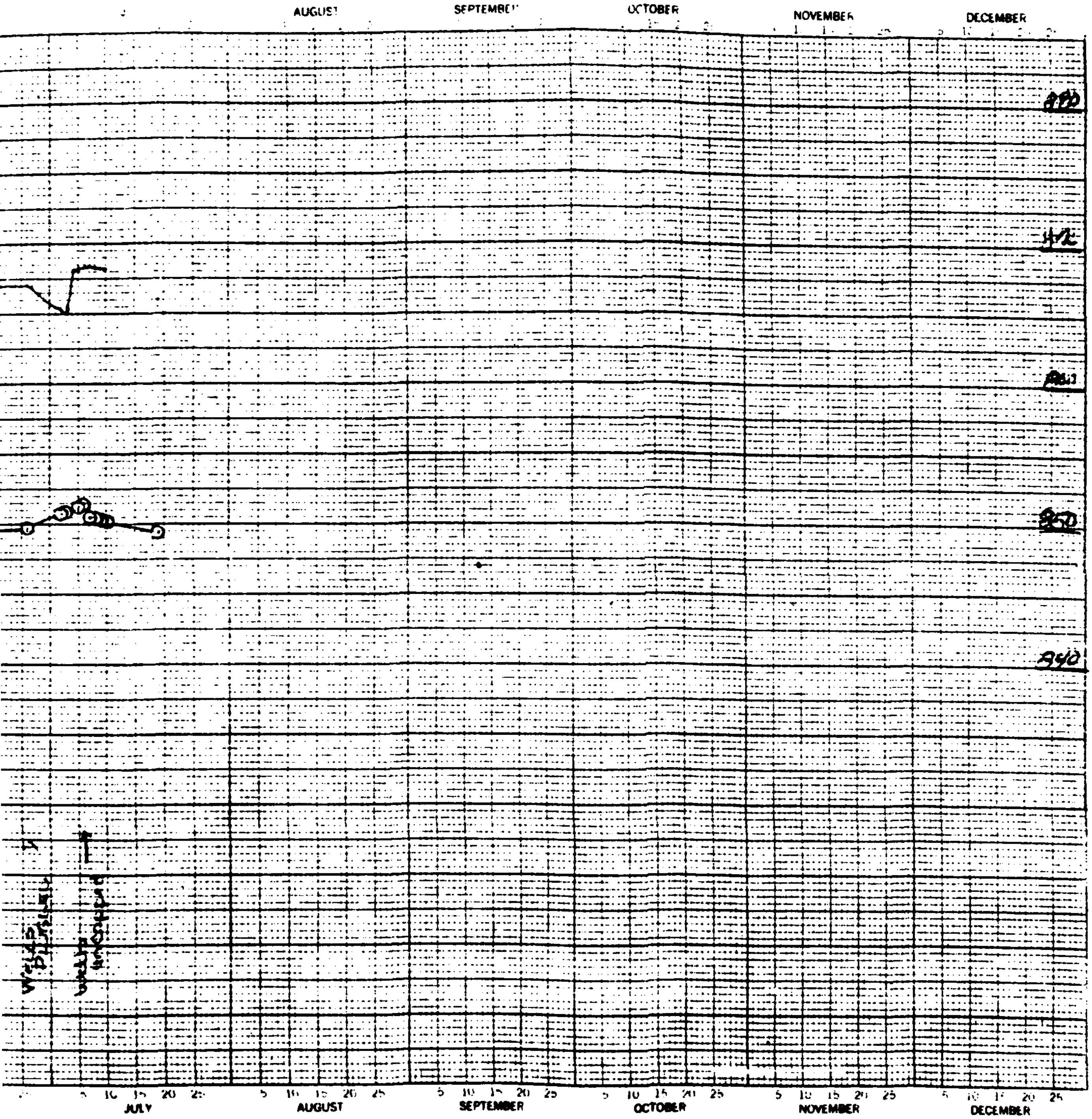
APPENDIX A | PLATE NO. 9



WELL INSTALLATION DATES	
WELL NO.	DATE INSTALLED
1	4-30-84
2	5-2-84
3	5-8-84
4	5-23-84
5	6-6-84
6	6-8-84
7	6-9-84

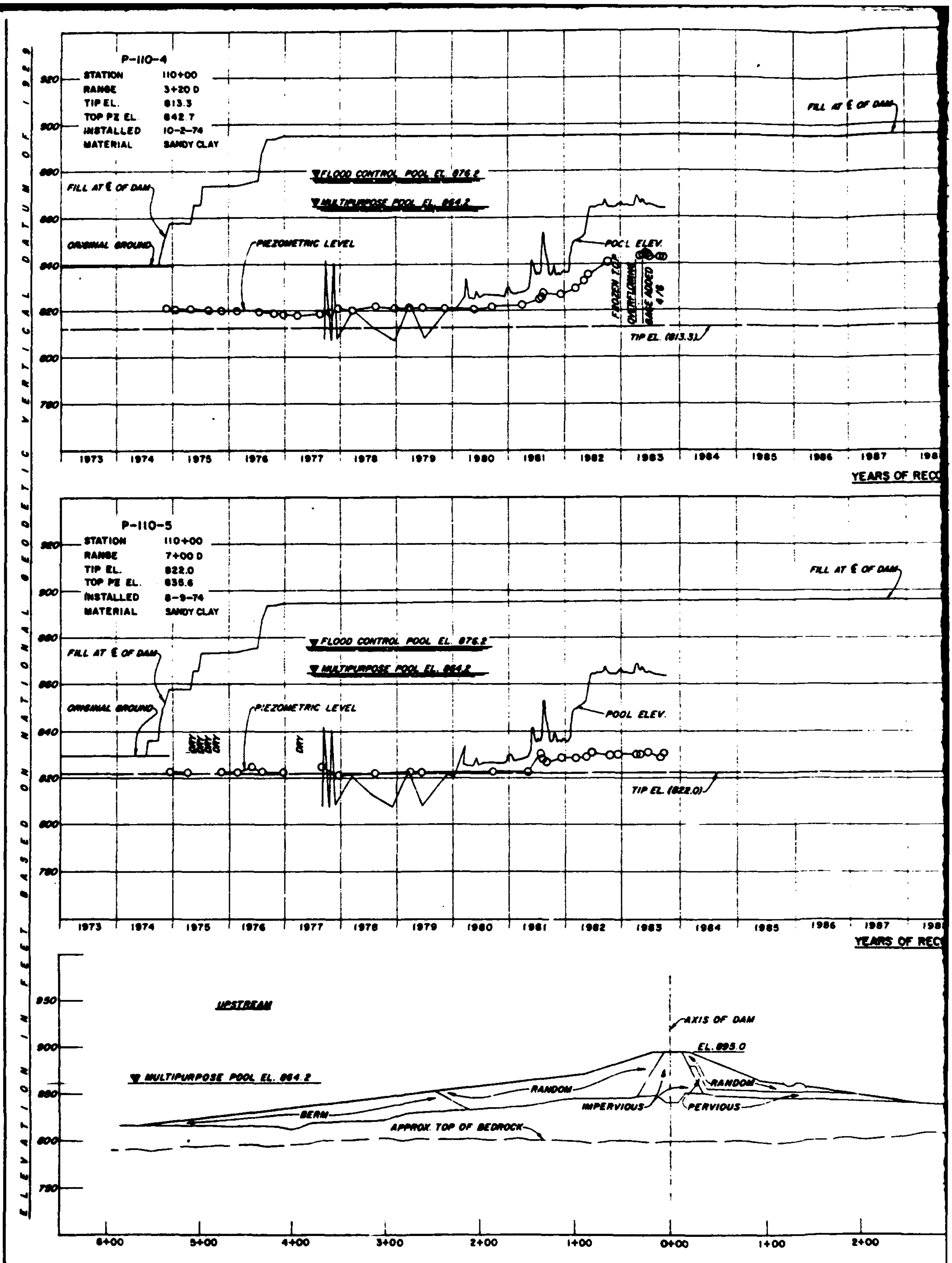
P-110-3

15-1-7

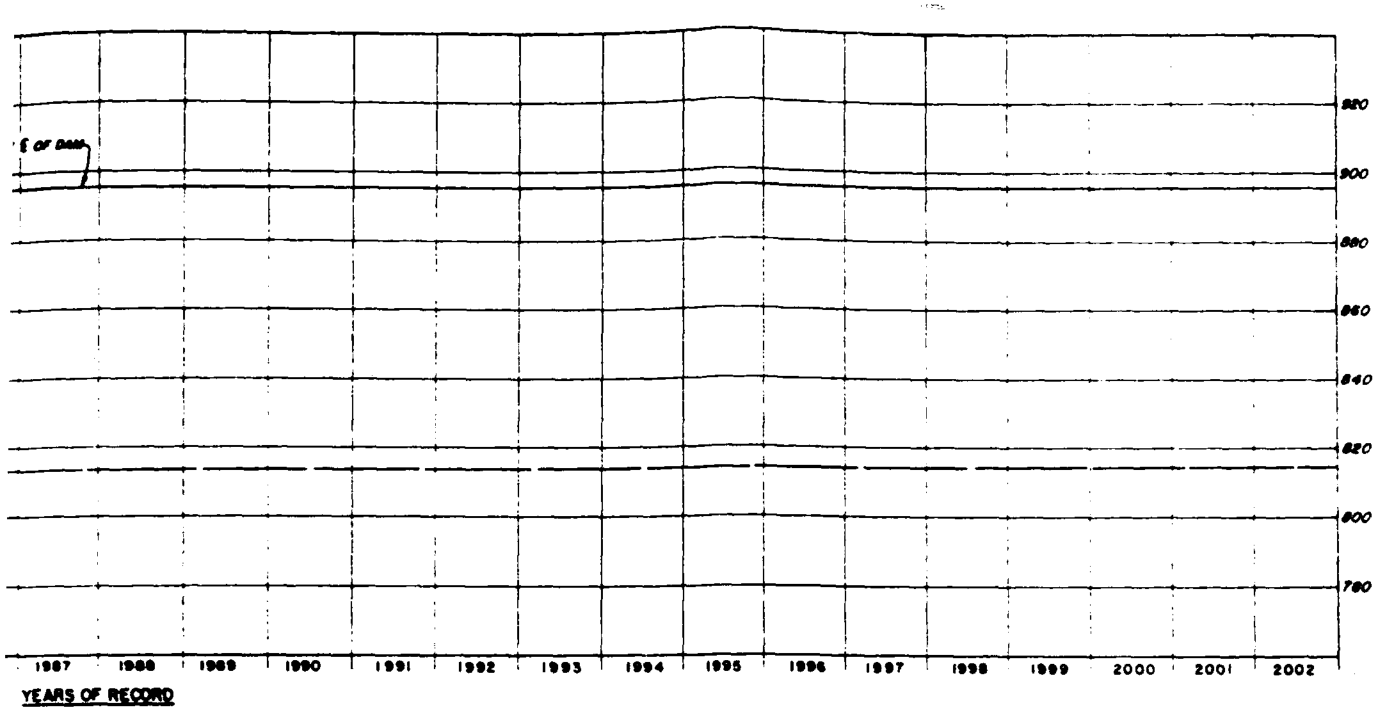


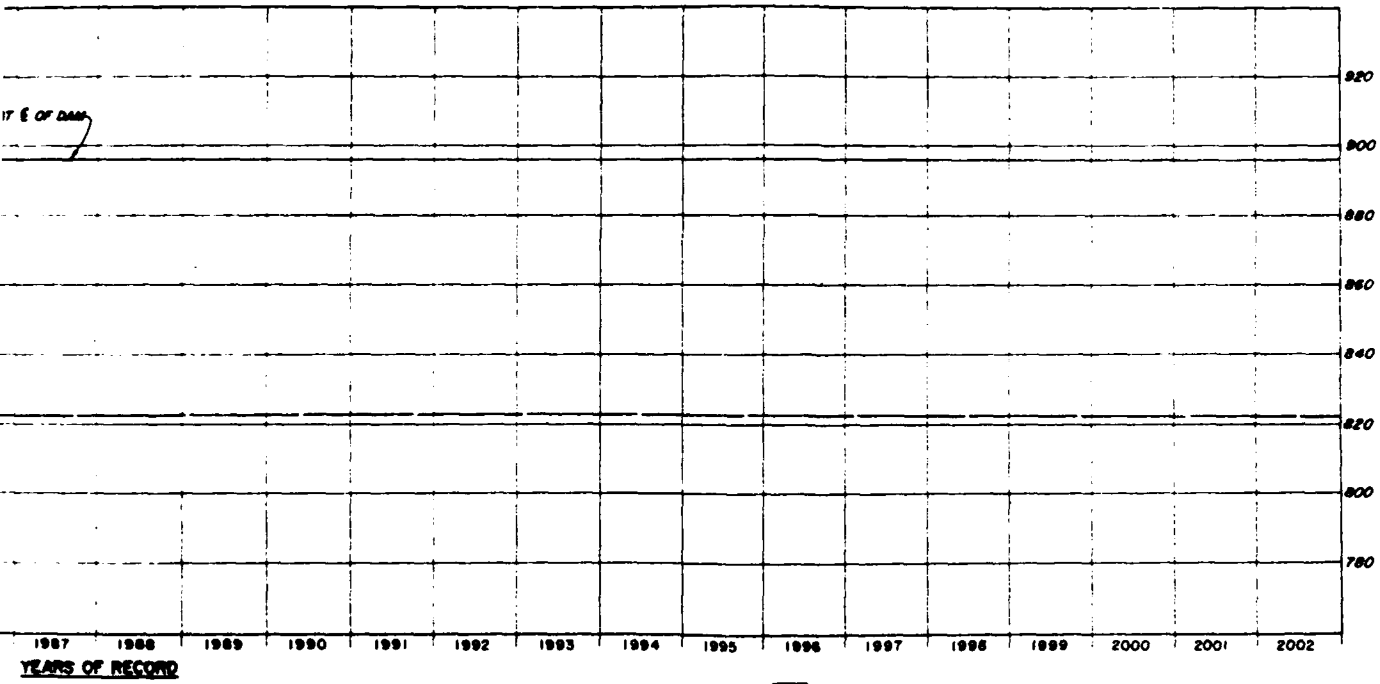
Sta 110+00
Rg 0+20 US

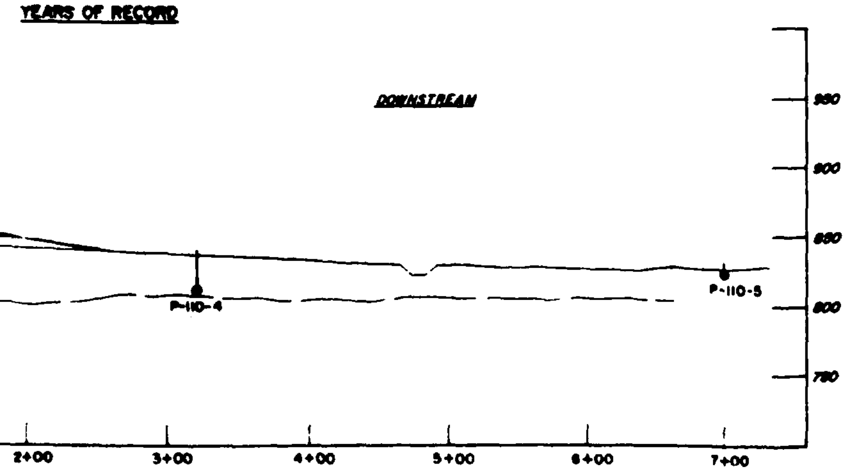
Smithville Dam
P-110-3
1984 PLOT
RP-3-1681
APPENDIX A PLATE 9A


ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929
P-110-4
STATION 110+00
RANGE 3+20 D
TIP EL. 813.3
TOP PZ EL. 842.7
INSTALLED 10-2-74
MATERIAL SANDY CLAY
FILL AT ℄ OF DAM
FLOOD CONTROL POOL EL. 876.2
MULTIPURPOSE POOL EL. 864.2
ORIGINAL GROUND
PIEZOMETRIC LEVEL
POOL ELEV.
FROZEN TOP
OVERFLOWING
GAGE ADDED 4/16
TIP EL. (813.3)
YEARS OF REC
P-110-5
STATION 110+00
RANGE 7+00 D
TIP EL. 822.0
TOP PZ EL. 835.6
INSTALLED 8-9-74
MATERIAL SANDY CLAY
FILL AT ℄ OF DAM
FLOOD CONTROL POOL EL. 876.2
MULTIPURPOSE POOL EL. 864.2
ORIGINAL GROUND
PIEZOMETRIC LEVEL
POOL ELEV.
DRY
TIP EL. (822.0)
YEARS OF REC
UPSTREAM
AXIS OF DAM
EL. 895.0
MULTIPURPOSE POOL EL. 864.2
RANDOM
IMPERVIOUS
PERVIOUS
BERM
APPROX. TOP OF BEDROCK

5-1-7







Revised September 1982

LITTLE PLATTE RIVER, MISSOURI

SMITHVILLE LAKE

PERIODIC INSPECTION

PIEZOMETRIC LEVELS
P-110-4 (OPEN TUBE)
P-110-5 (OPEN TUBE)

in 1 sheet — Sheet No. 1 — Scale: as shown

CORPS OF ENGINEERS U. S. ARMY

KANSAS CITY DISTRICT

FILE NO RP-3-[redacted] 1682

MARCH 1981

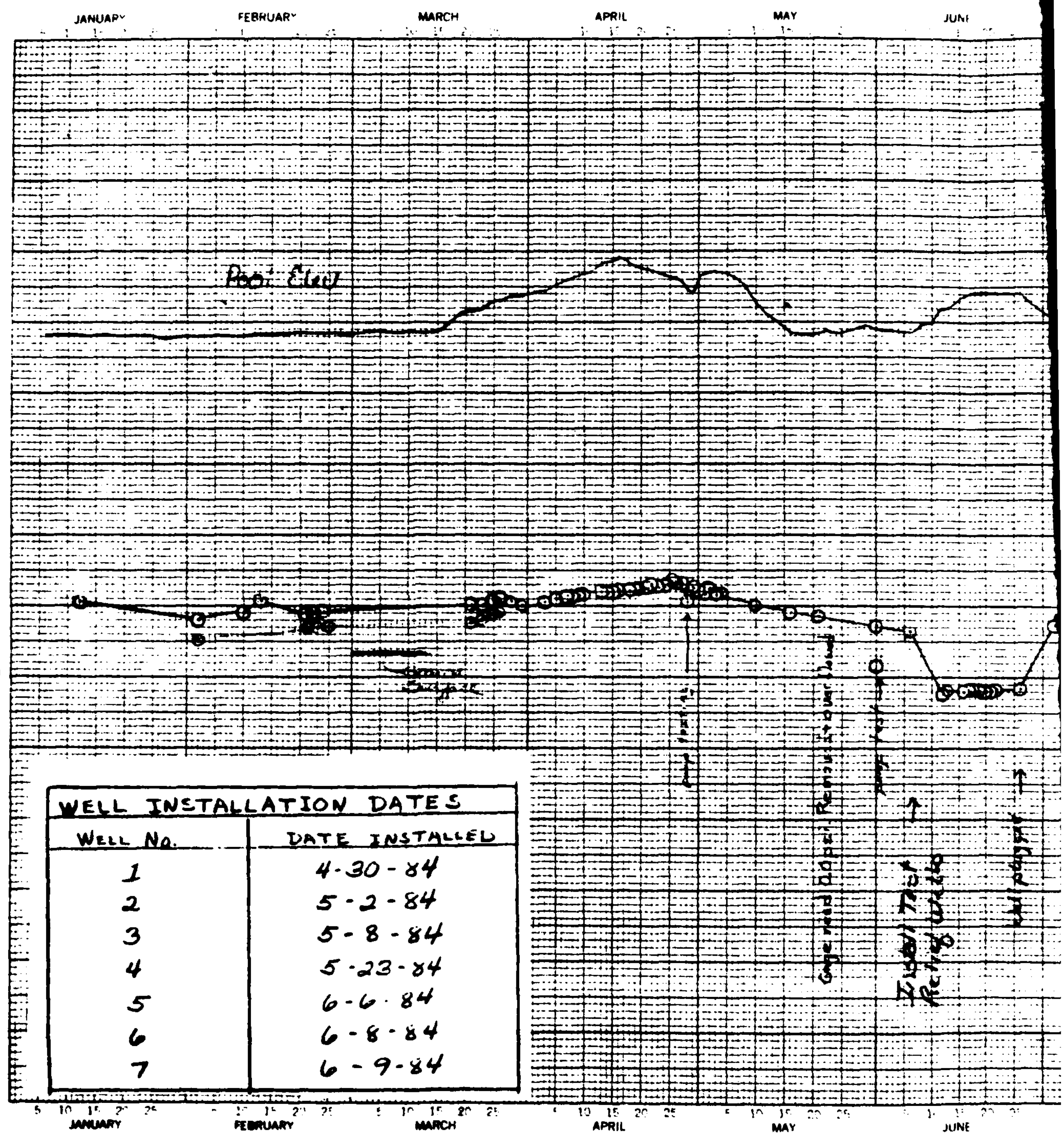
APPENDIX A PLATE NO. 10

1

2

47 2813

K•E 1 YEAR BY DAYS X 150 DIVISIONS
KEUFFEL & ESSER CO.

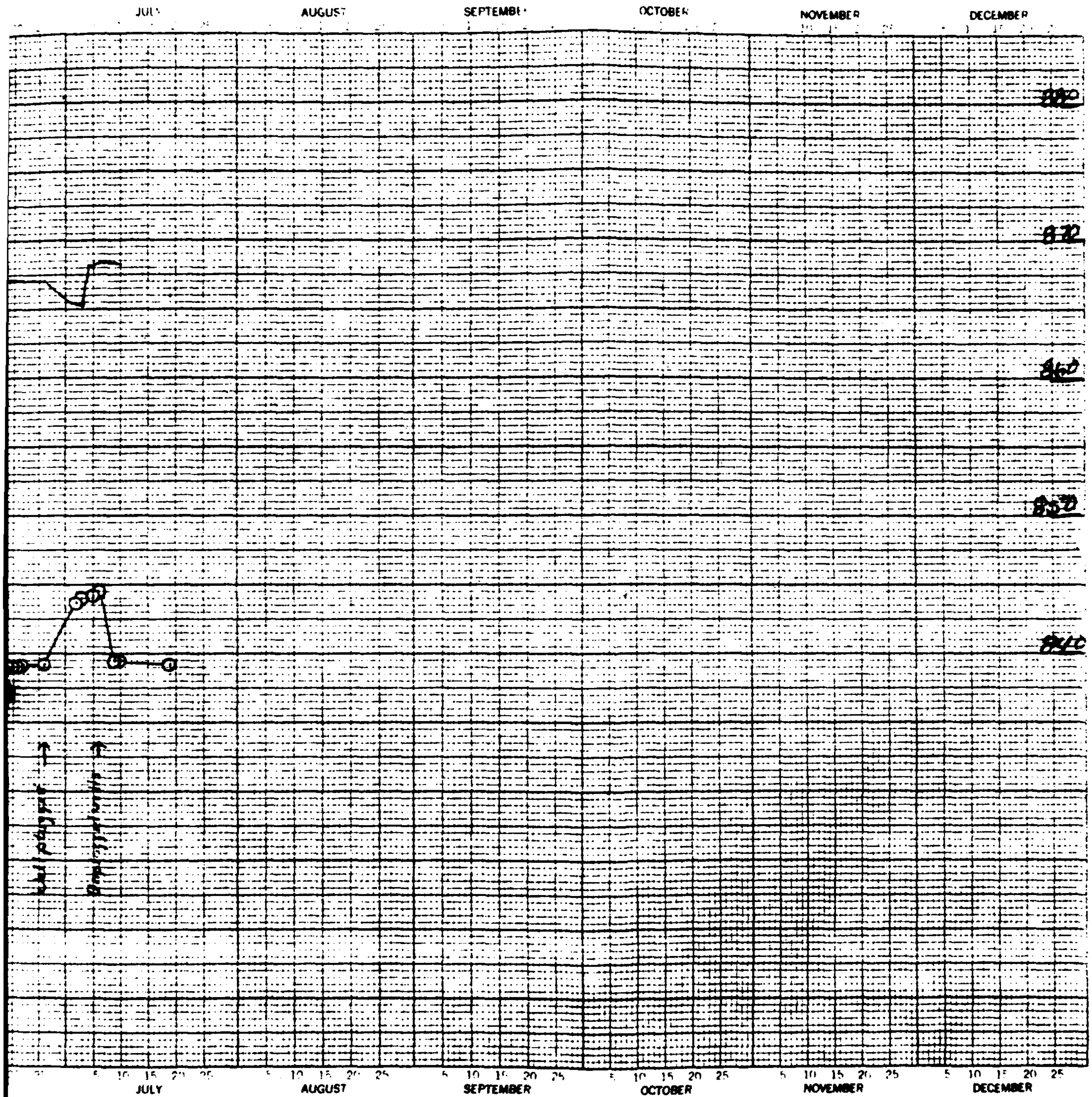


WELL INSTALLATION DATES

WELL NO.	DATE INSTALLED
1	4-30-84
2	5-2-84
3	5-8-84
4	5-23-84
5	6-6-84
6	6-8-84
7	6-9-84

P-110-4

15-1-7



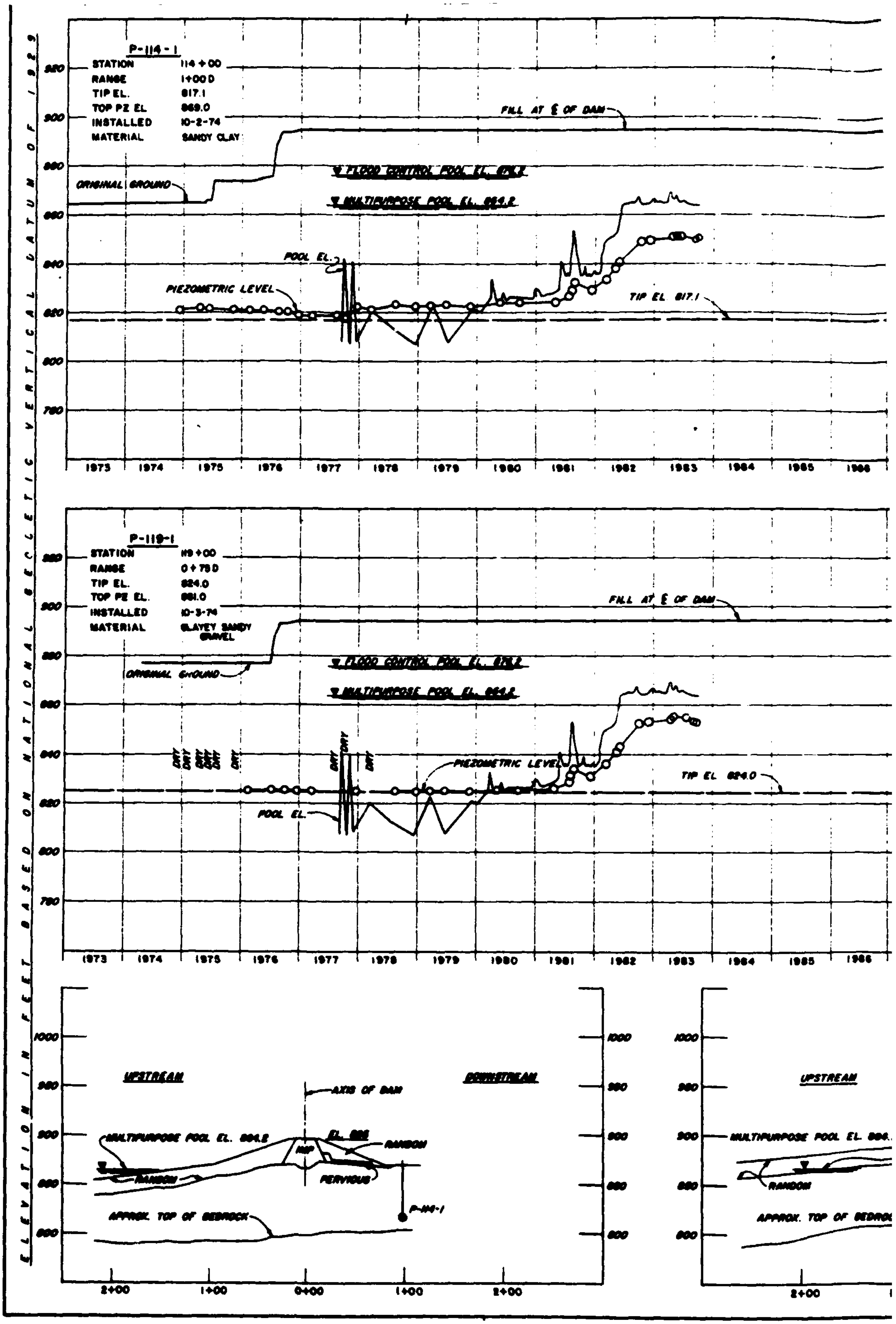
Sta 110+00
R 3+20 D/S

O Readings changed to correlate w/ 5 lb. gage. 15# gage and diesel removed 3-27-84. Replaced w/ 5# Marshall [illegible] gage.

RP-3-1683

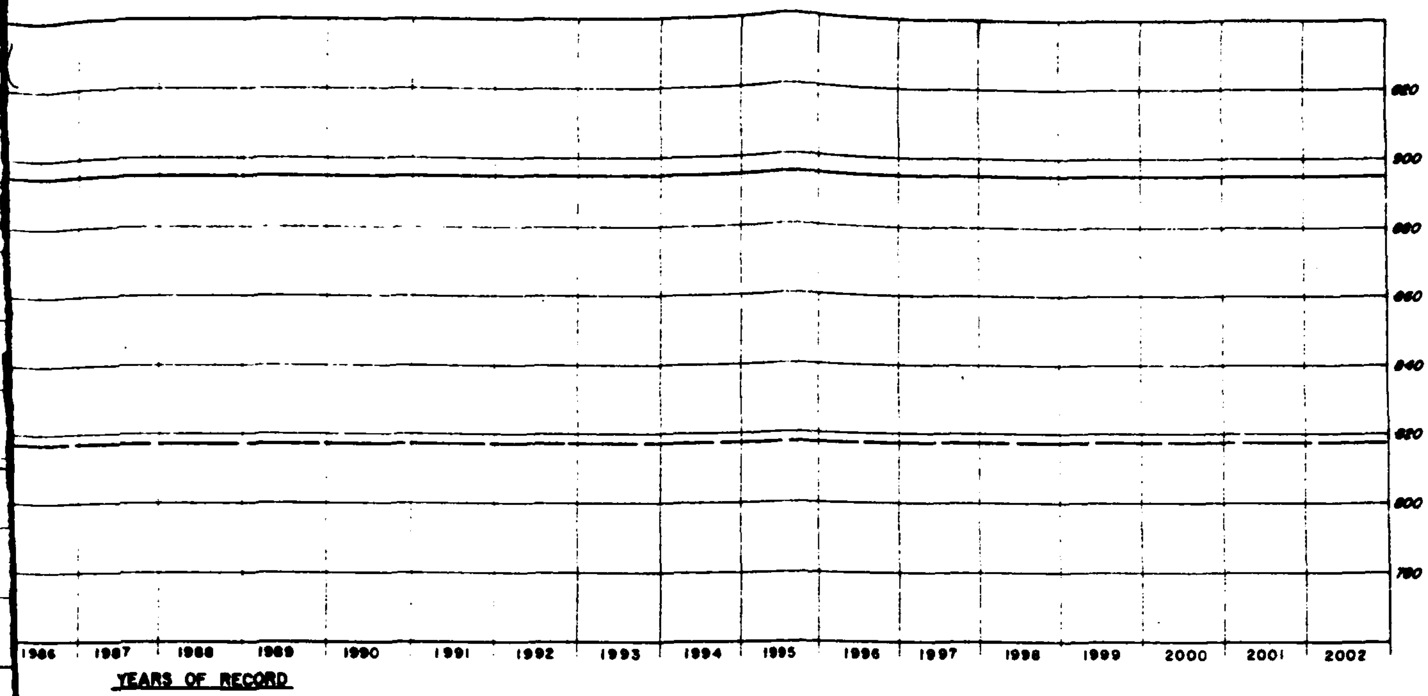
APPENDIX A PLATE 10A

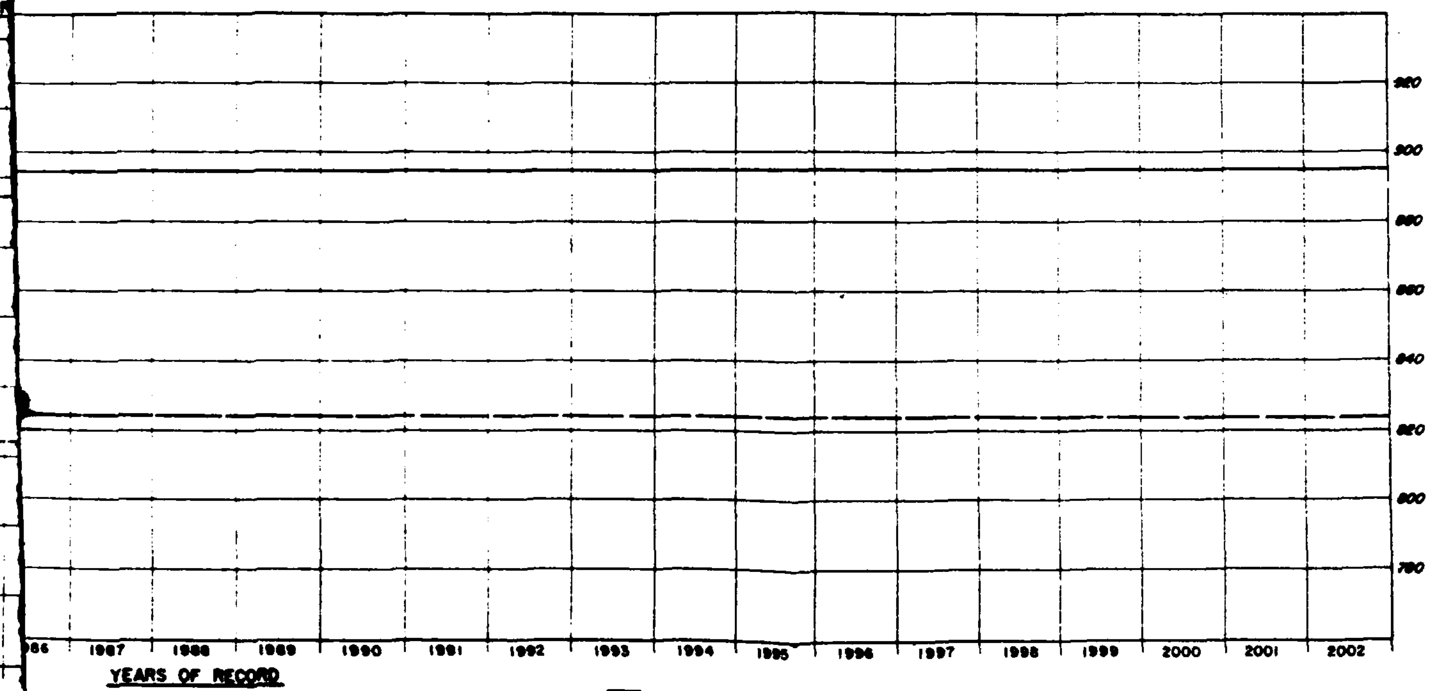
1

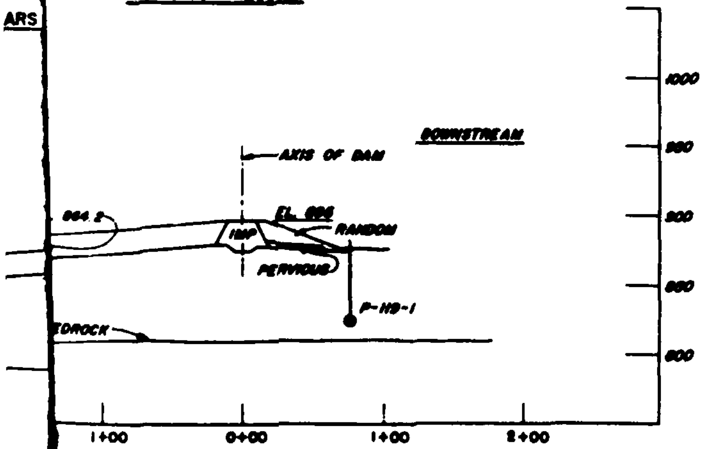


ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929
P-114-1
STATION 114+00
RANGE 1+00D
TIP EL. 817.1
TOP PZ EL. 869.0
INSTALLED 10-2-74
MATERIAL SANDY CLAY
FILL AT ℄ OF DAM
ORIGINAL GROUND
FLOOD CONTROL POOL EL. 876.2
MULTIPURPOSE POOL EL. 864.2
POOL EL.
PIEZOMETRIC LEVEL
TIP EL. 817.1
P-119-1
STATION 119+00
RANGE 0+75D
TIP EL. 824.0
TOP PZ EL. 881.0
INSTALLED 10-3-74
MATERIAL CLAYEY SANDY GRAVEL
ORIGINAL GROUND
DRY
TIP EL. 824.0
1973 1974 1975 1976 1977 1978 1979 1980 1981 1982 1983 1984 1985 1986
UPSTREAM
DOWNSTREAM
AXIS OF DAM
MULTIPURPOSE POOL EL. 864.2
EL. 896
RANDOM
IMP
PERVIOUS
P-114-1
APPROX. TOP OF BEDROCK
2+00 1+00 0+00 1+00 2+00

15-1-7







Revised September 1982

LITTLE PLATTE RIVER, MISSOURI

SMITHVILLE LAKE

PERIODIC INSPECTION

PIEZOMETRIC LEVELS
P-114-1(OPEN TUBE)
P-119-1(OPEN TUBE)

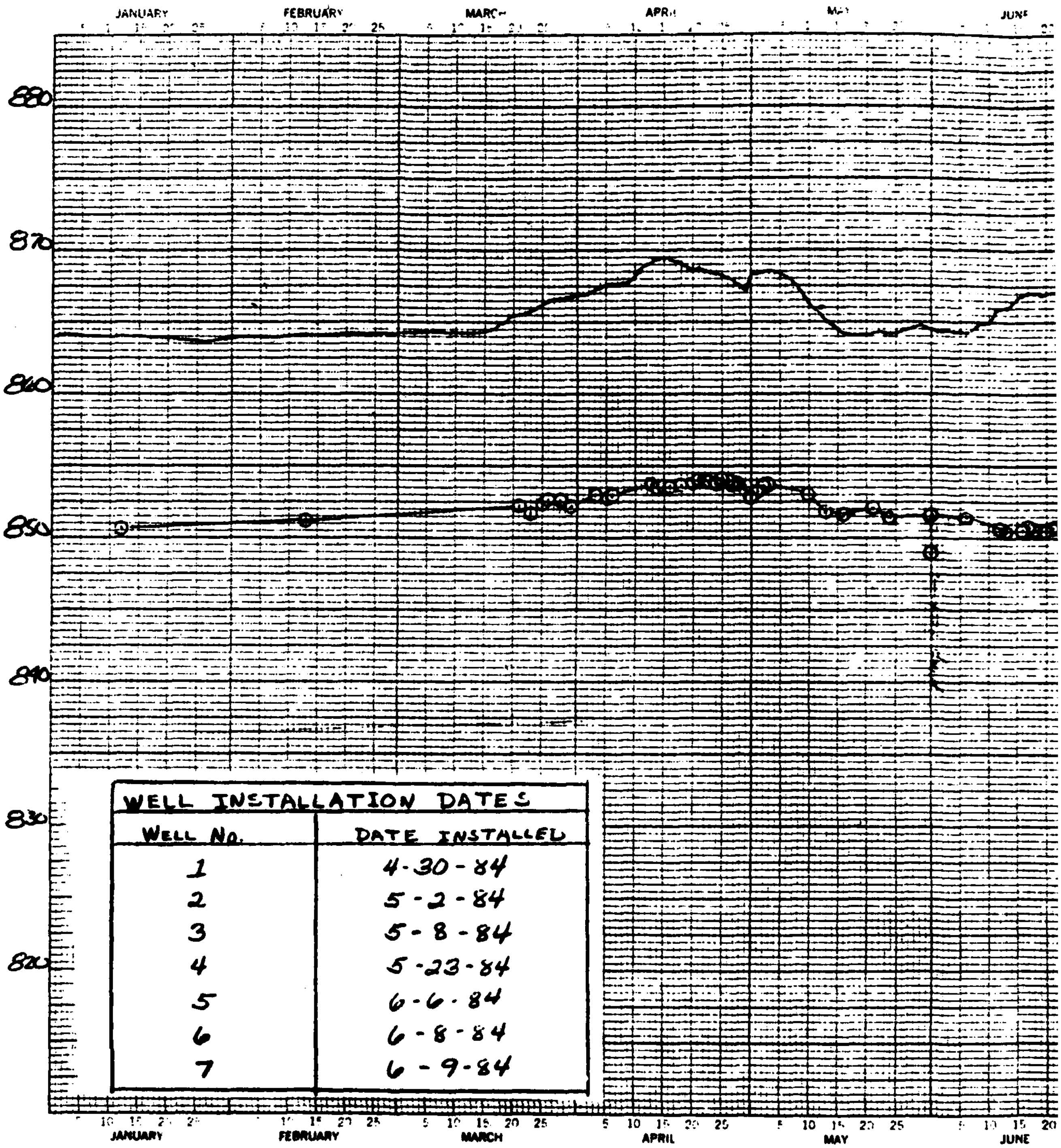
In 1 sheet | Sheet No. 1 | Scale: as shown

CORPS OF ENGINEERS U. S. ARMY
KANSAS CITY DISTRICT

FILE NO. RP-3-[redacted] 1684

MARCH 1981

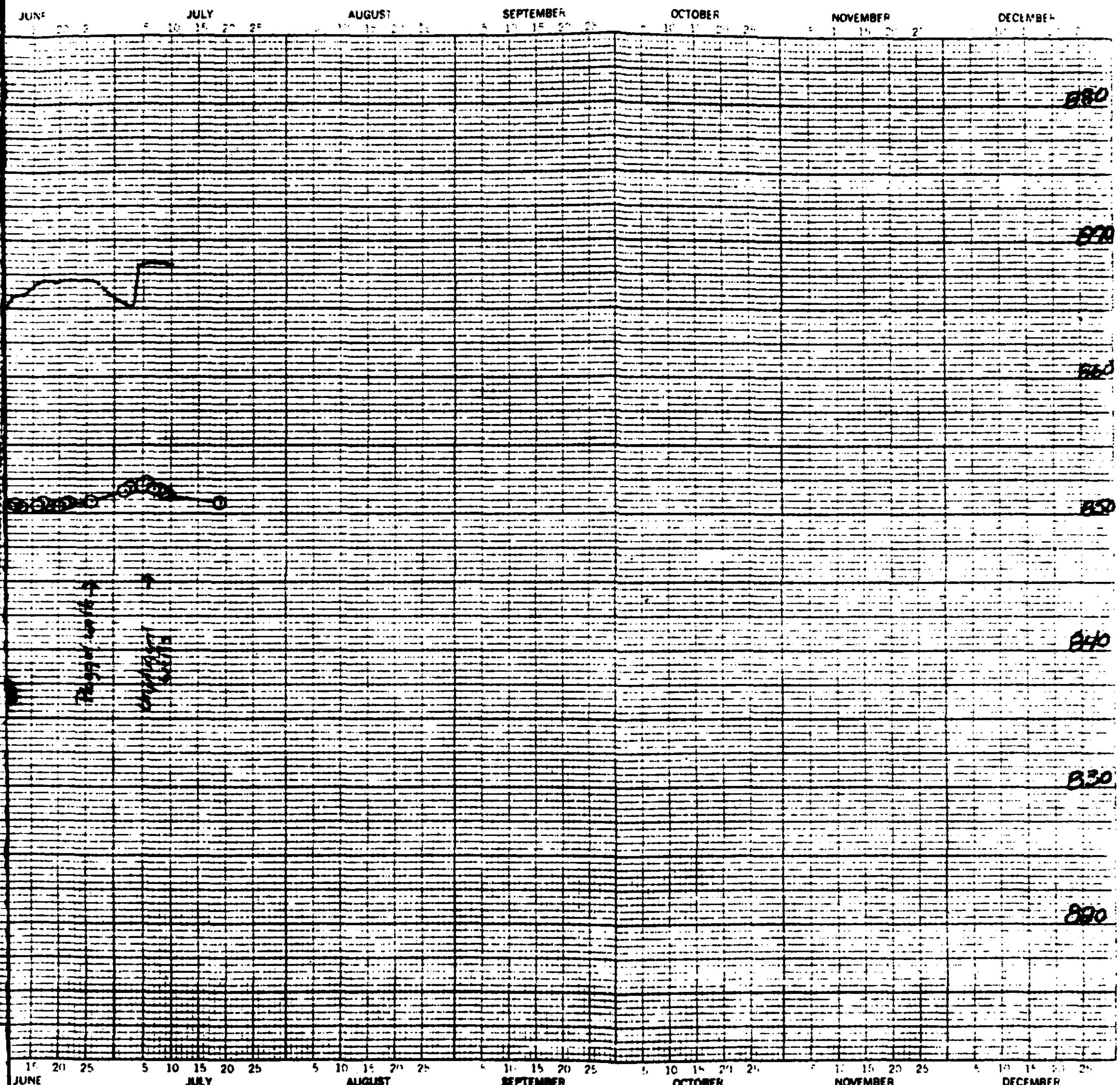
APPENDIX A PLATE NO. 11



WELL INSTALLATION DATES

WELL NO.	DATE INSTALLED
1	4-30-84
2	5-2-84
3	5-8-84
4	5-23-84
5	6-6-84
6	6-8-84
7	6-9-84

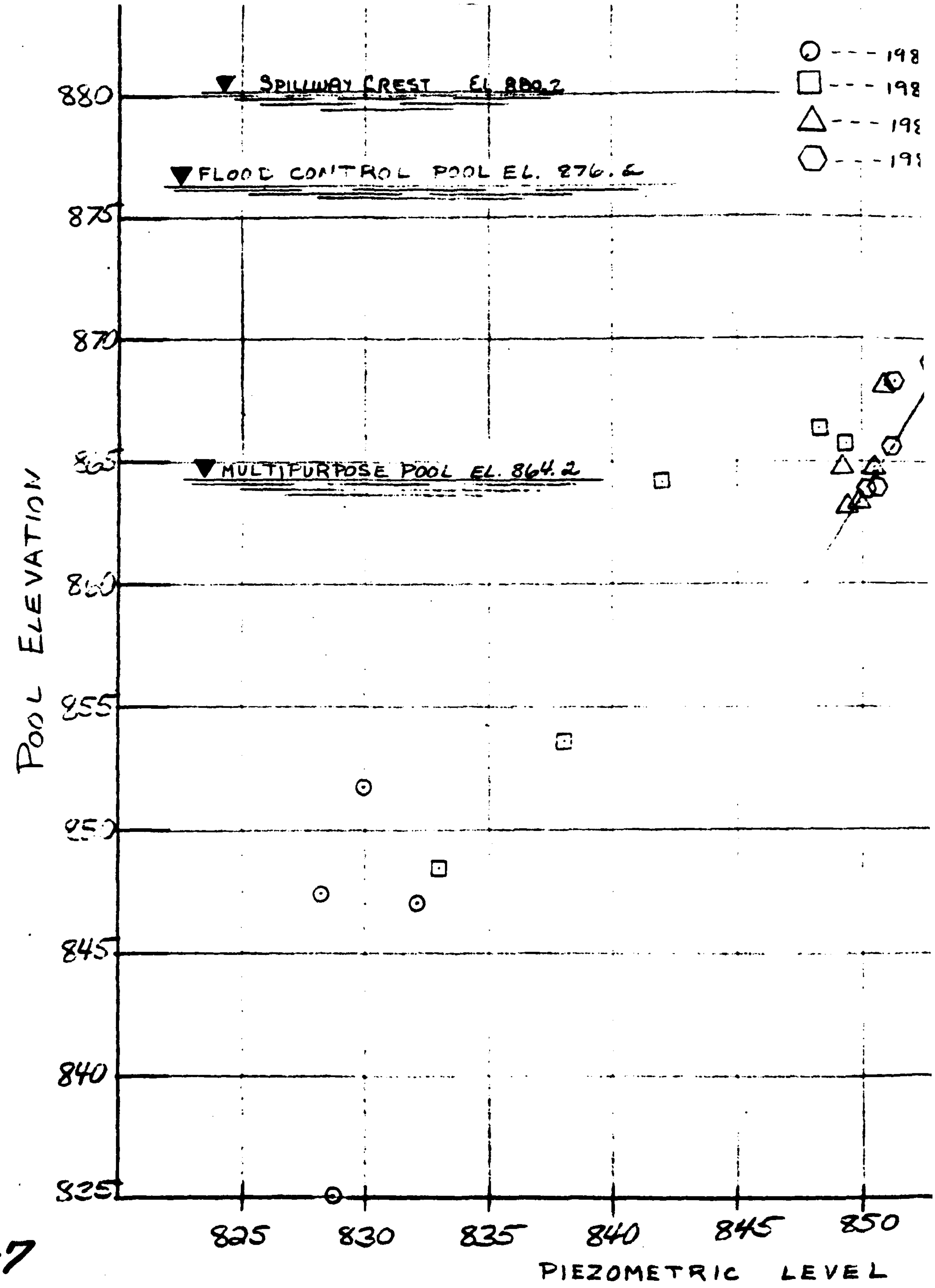
15-1-7



STA 114+00
K 1+00 D/S

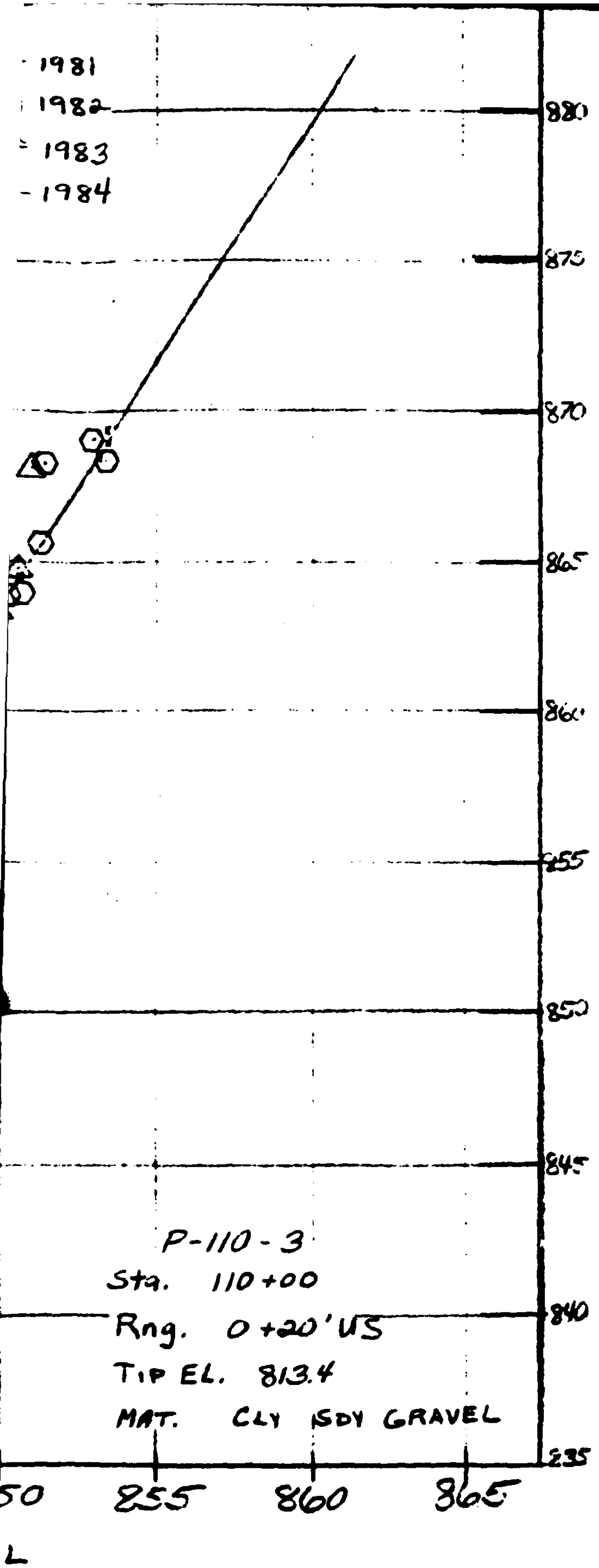
SMITHVILLE DAM
P-114-1

RP-3-1685
APPENDIX A PLATE 11A



SPILLWAY CREST EL 880.2
FLOOD CONTROL POOL EL. 876.6
MULTIPURPOSE POOL EL. 864.2
POOL ELEVATION
880
875
870
865
860
855
850
845
840
835
825
830
835
840
845
850
PIEZOMETRIC LEVEL

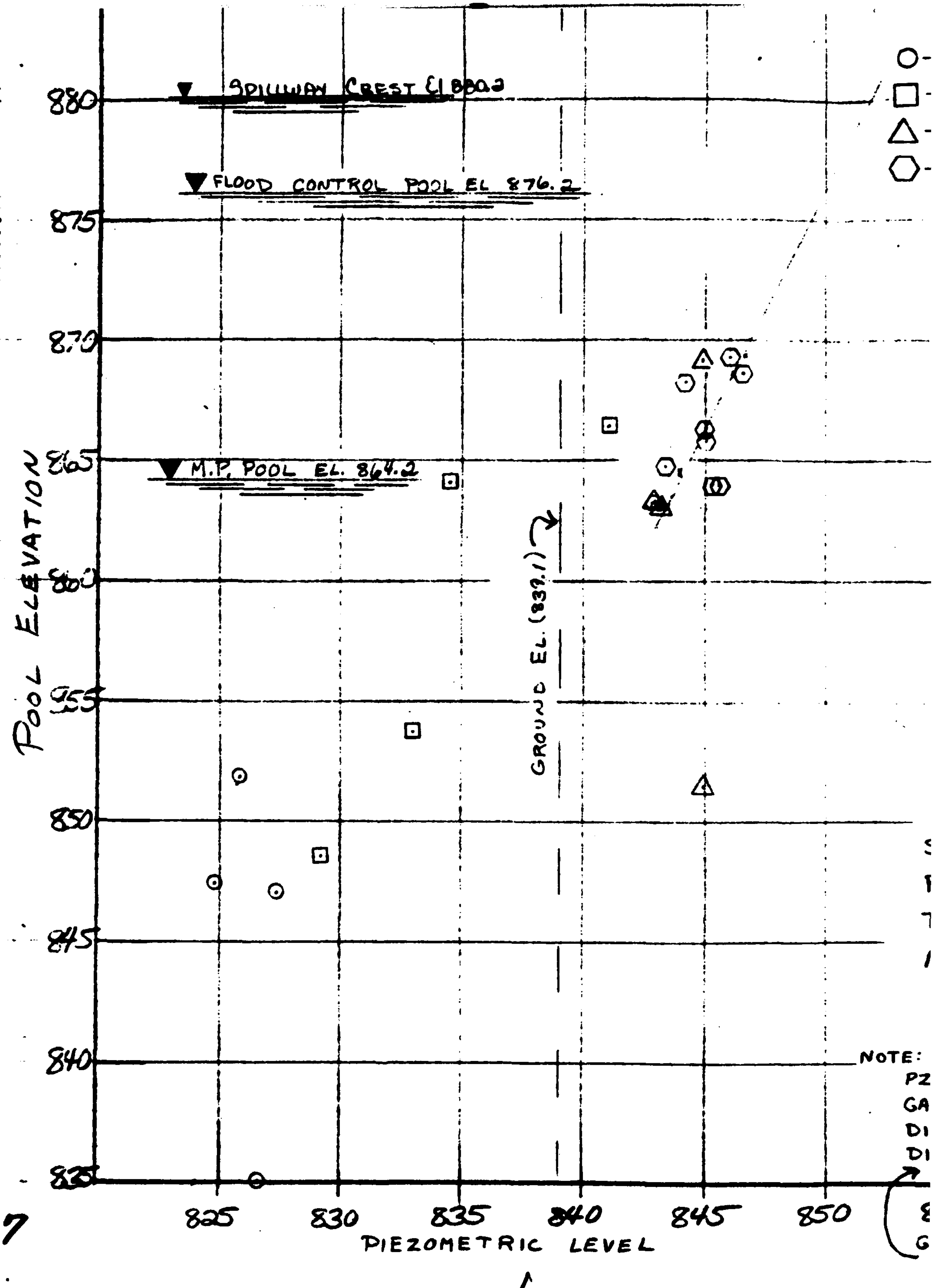
15-1-7



SMITHVILLE DAM
LEFT ABUTMENT

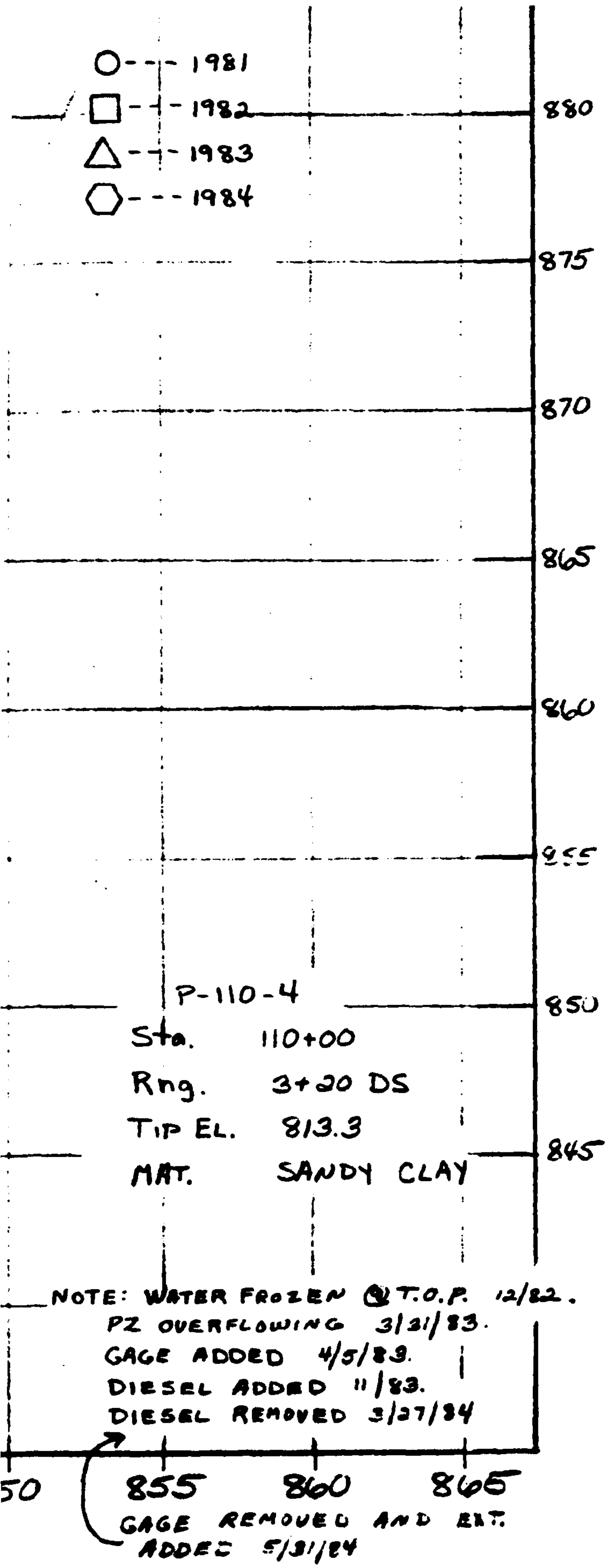
Pz vs Pool Plot
P-110-3

APPENDIX A RP-3-1686
PLATE 12 JULY 1984



SPILLWAY CREST El 880.2
FLOOD CONTROL POOL EL 876.2
M.P. POOL EL. 864.2
GROUND EL. (839.1)
POOL ELEVATION
PIEZOMETRIC LEVEL
880
875
870
865
860
850
845
840
825
830
835
840
845
850
NOTE:

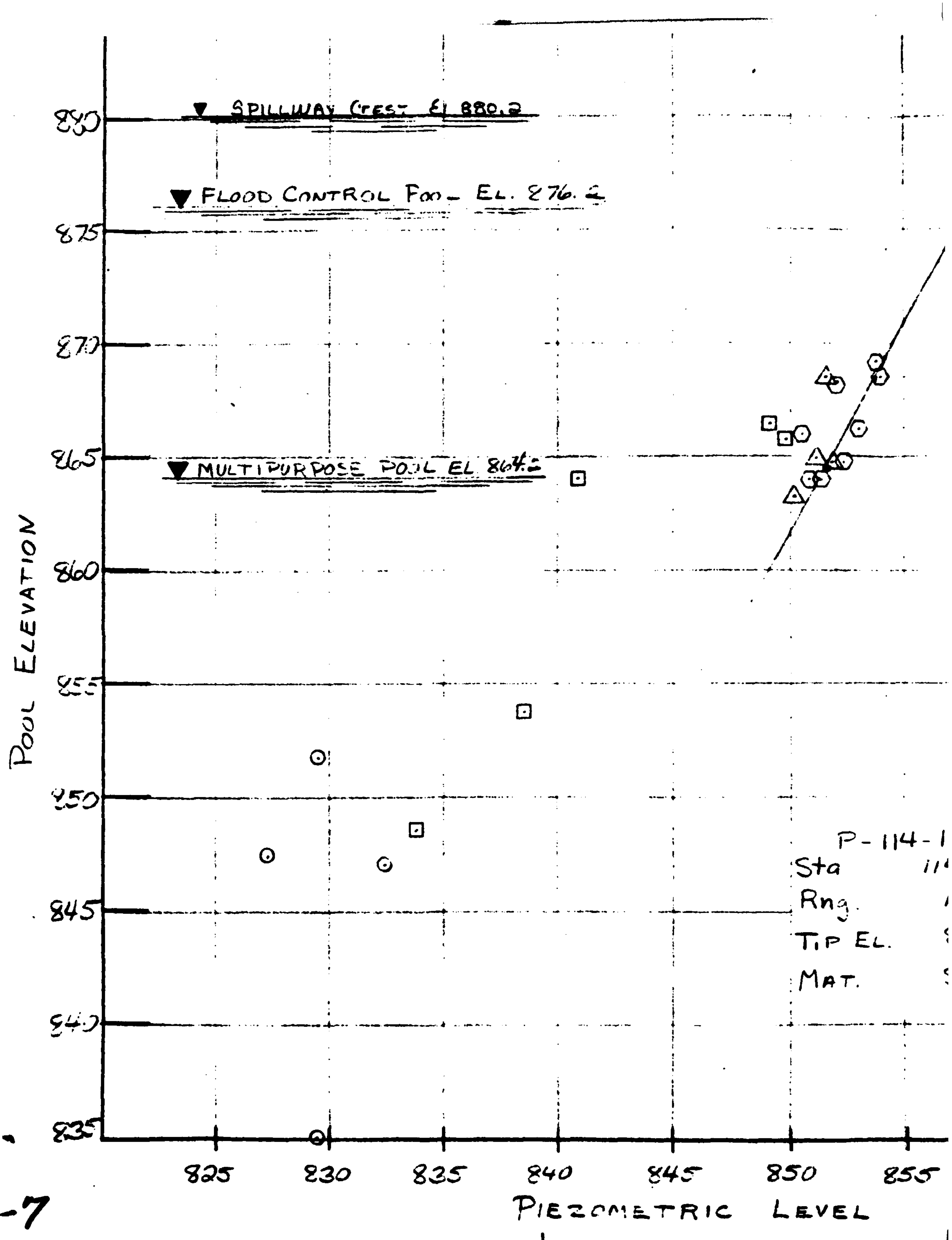
15-1-7



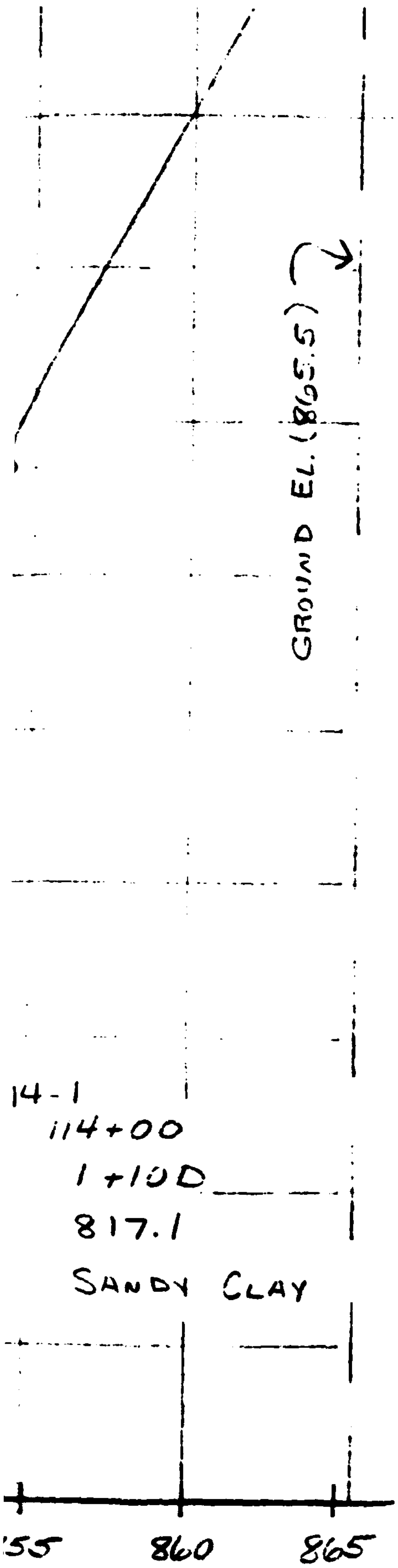
Smithville Dam
Left Abutment

Pz vs Pool Plot
P-110-4

APPENDIX A RP-3-1687
PLATE 13 JULY 1984


SPILLWAY CREST EL 880.2
FLOOD CONTROL POOL EL. 876.2
MULTIPURPOSE POOL EL 864.2
POOL ELEVATION
880
875
870
865
860
855
850
845
840
835
825
830
835
840
845
850
855
PIEZOMETRIC LEVEL
P-114-1
Sta
Rng.
TIP EL.
MAT.

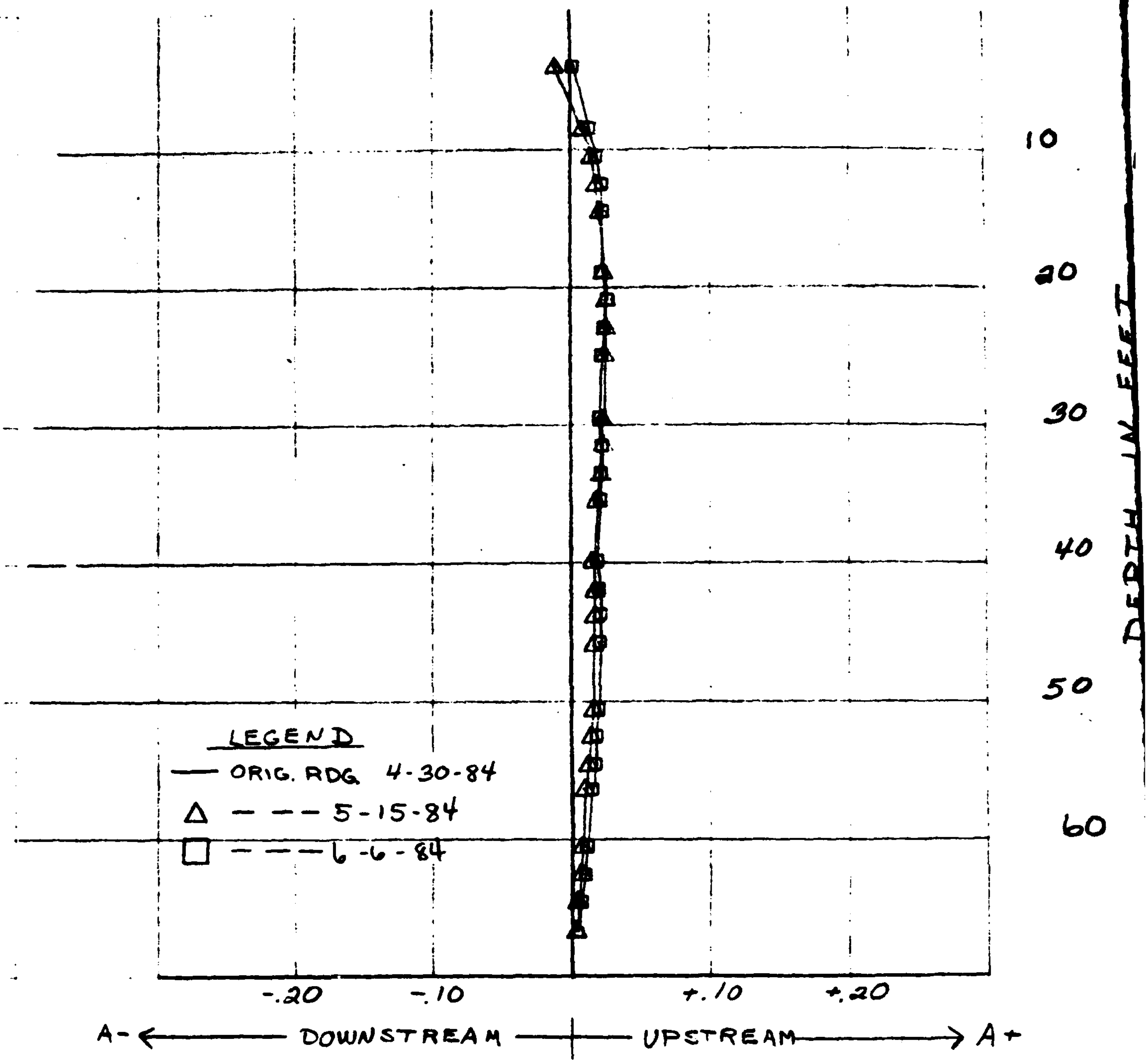
15-1-7



SMITHVILLE DAM
LEFT ABUTMENT

PZ VS POOL PLOT
P-114-1

APPENDIX A RP-3-1688
PLATE 14 JULY 1984

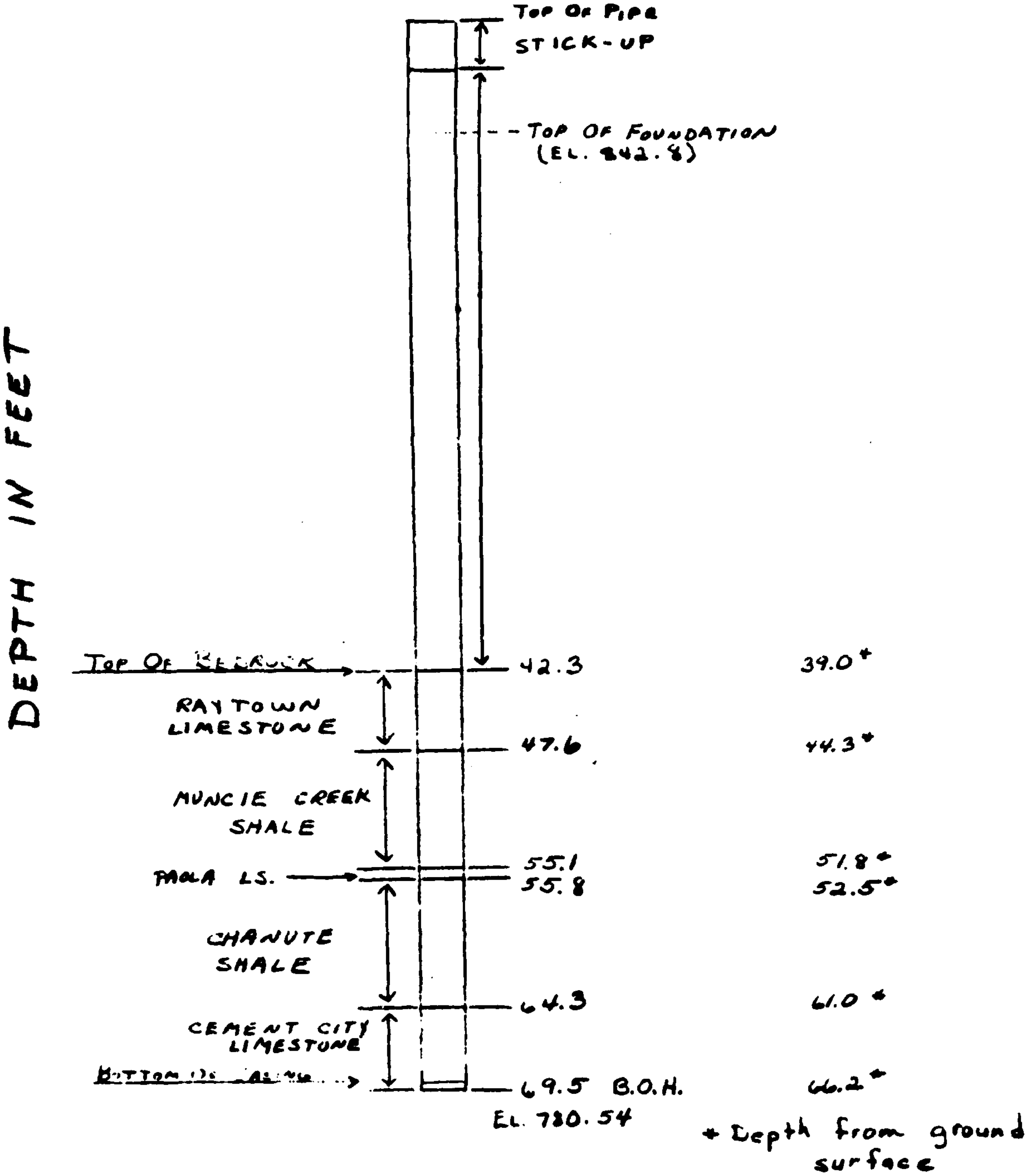


ACCUMULATED MOVEMENT PERPINDICULAR TO DAM AXIS

MOVEMENT IN INCHES

I-110-1
Sta. 110+00
Rng. 2+40 DS

15-1-7



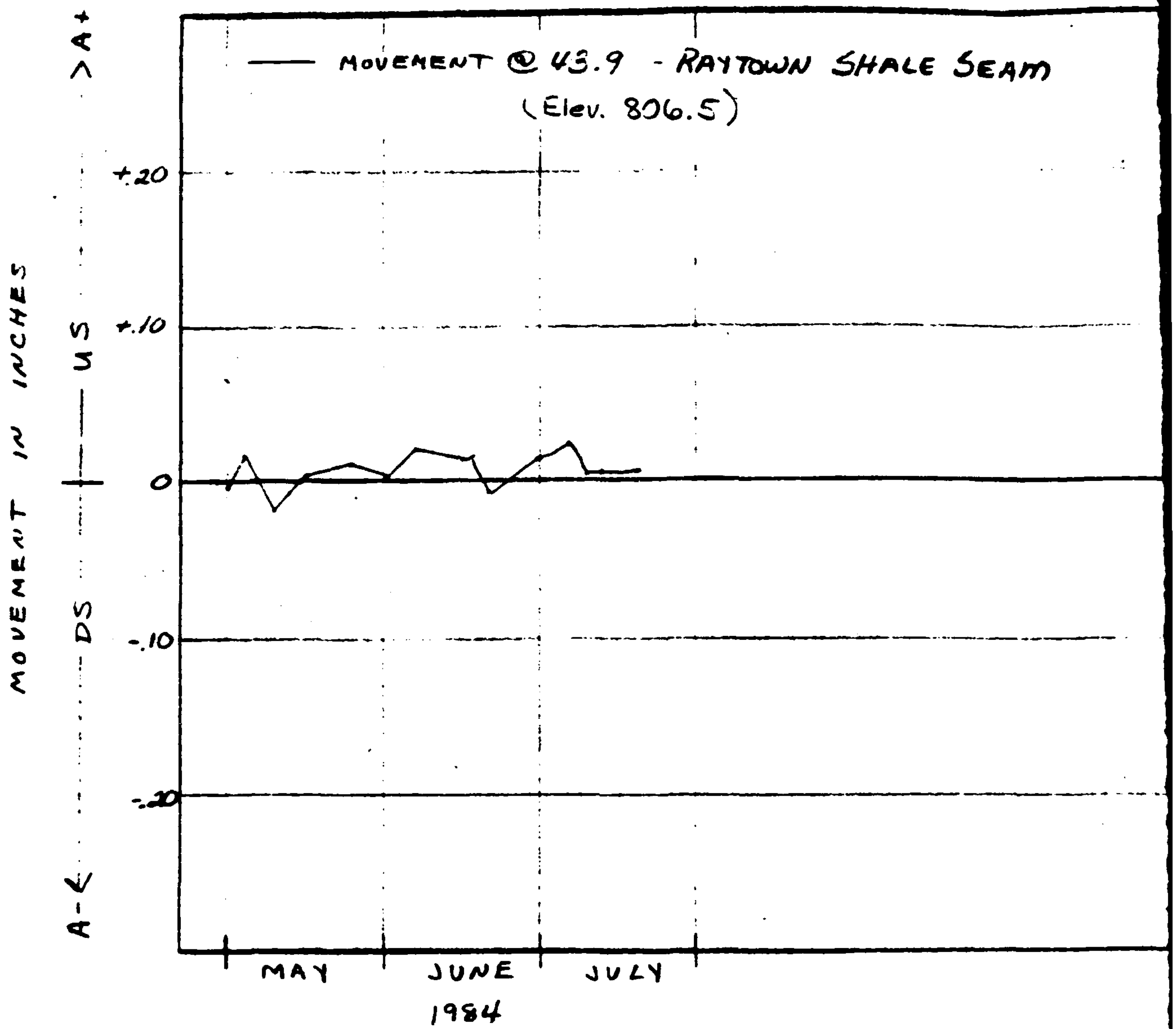
SMITHVILLE DAM
LEFT ABUTMENT

Inclinometer Plot
I-10-1

TYPICAL READINGS

APPENDIX A
PLATE 15

RP-3-1689
July 1984

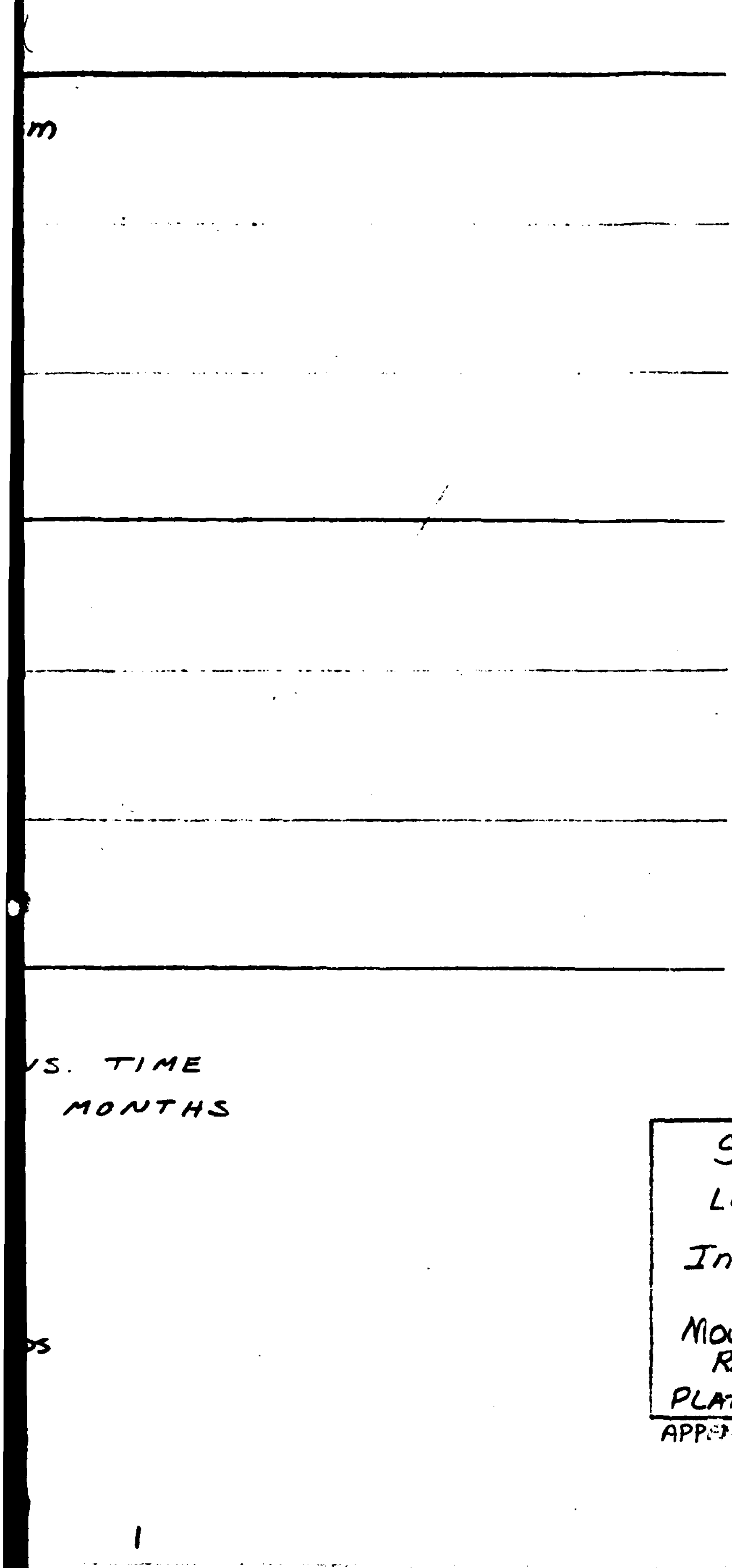


MOVEMENT VS. TI
TIME IN MON

I-110-1

Sta 110+00
Rg 2+40 DS

15-1-17



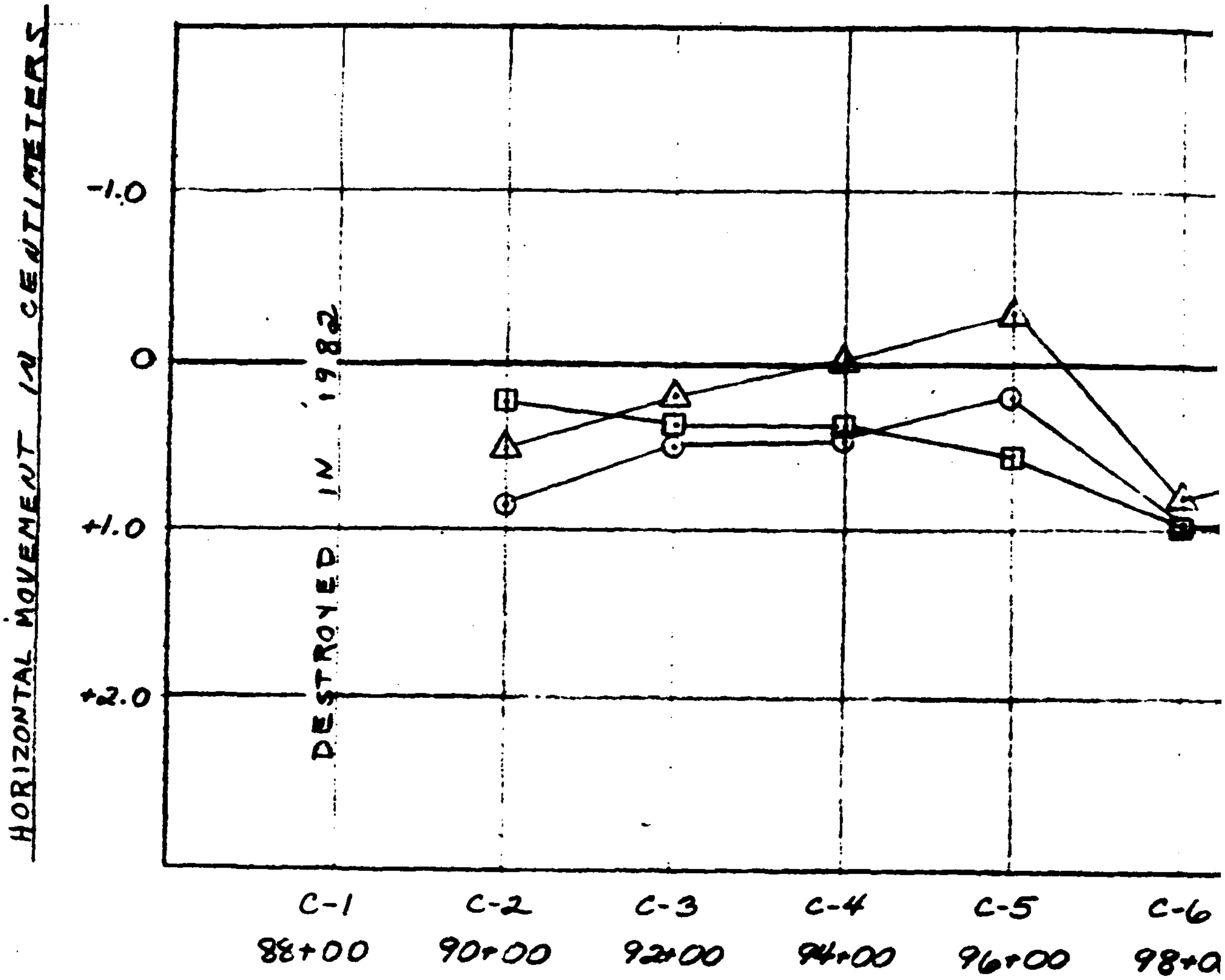
SMITHVILLE DAM
LEFT ABUTMENT

INCLINOMETER PLOT
I-110-1

MOVEMENT vs TIME at
RAYTOWN SHALE SEAM

PLATE 16 JULY 1984

APPENDIX A RP-3-1690

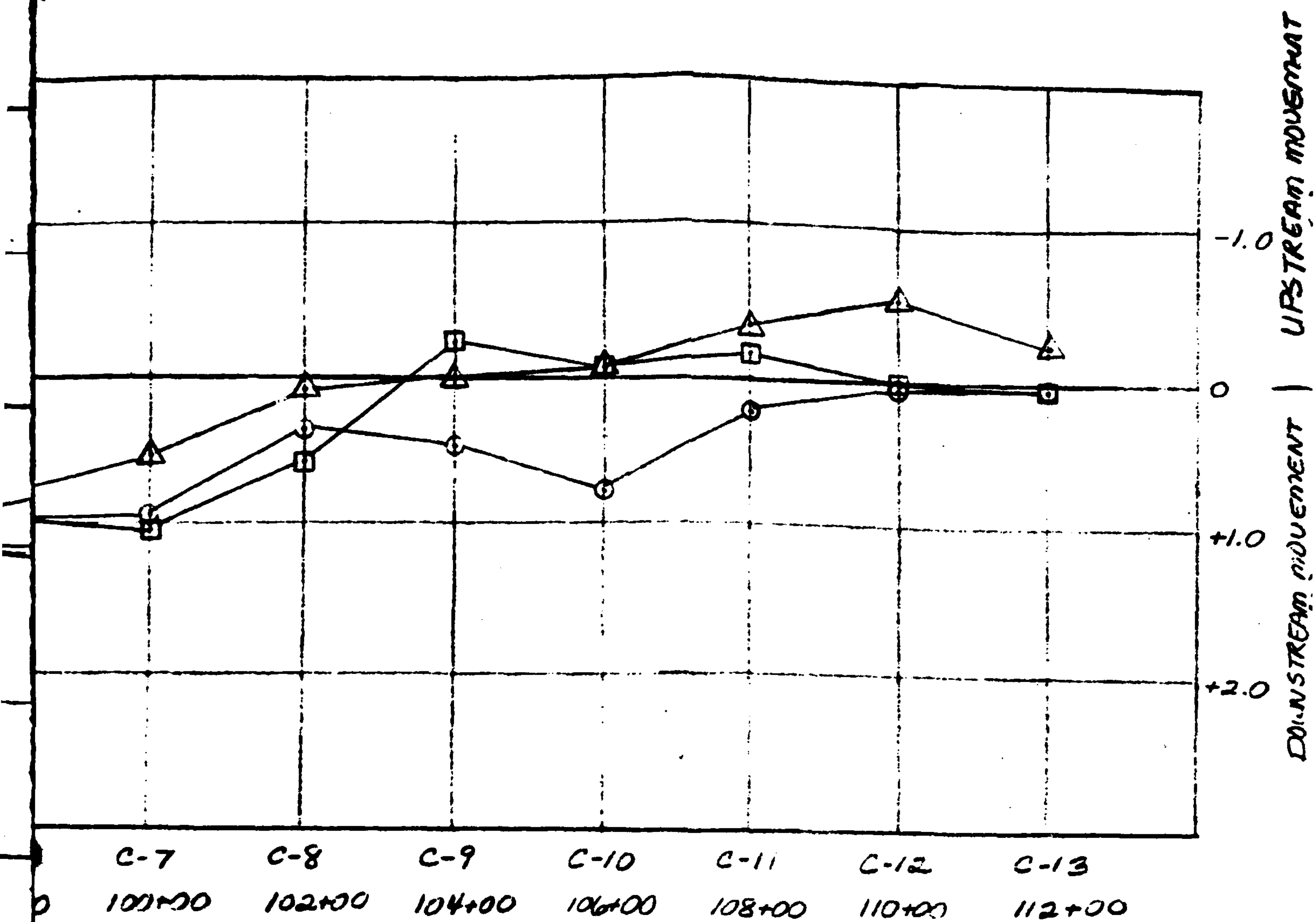


ALINEMEN

RA

15-1-7

1



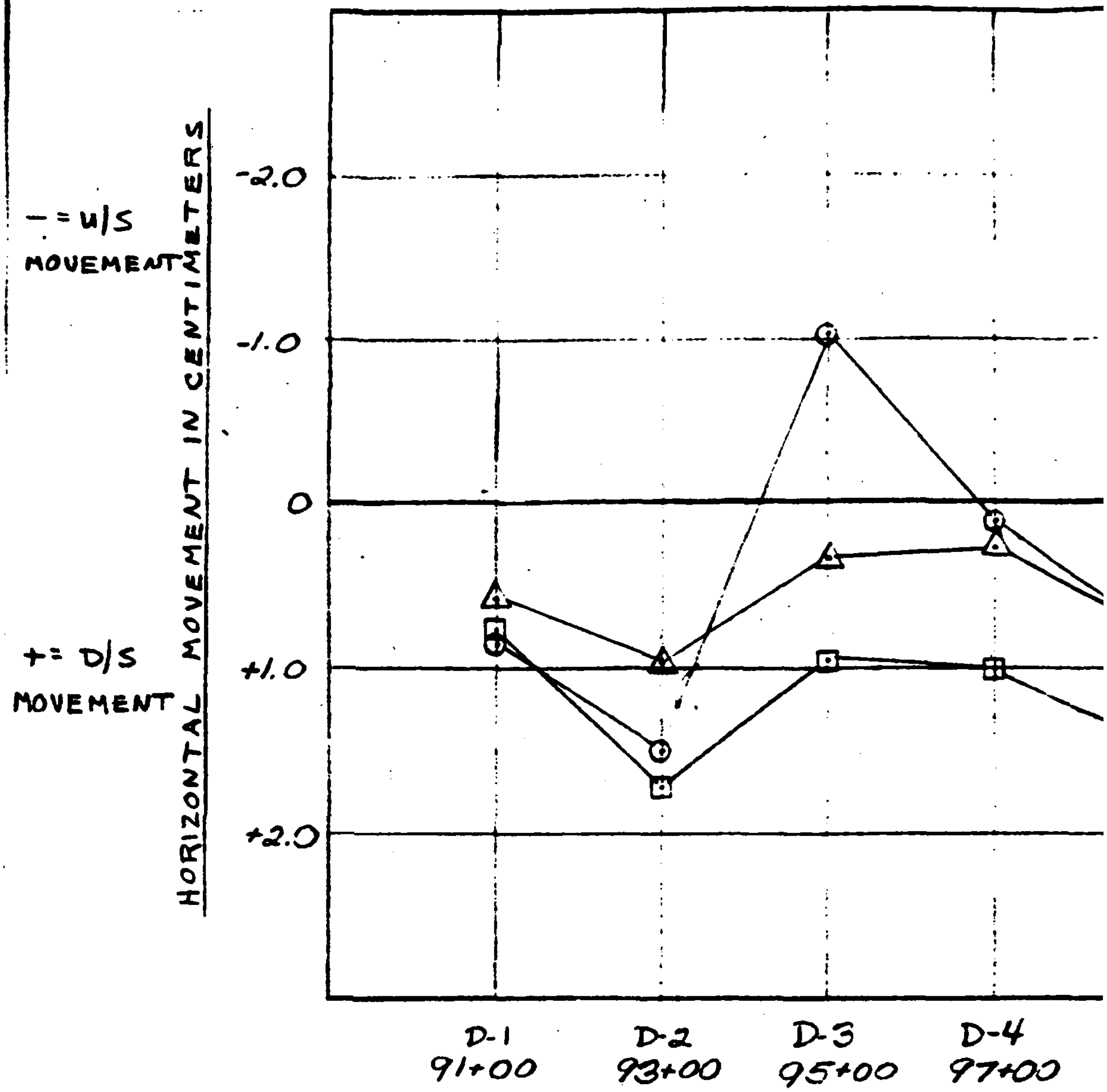
T MONUMENT LINE "C"

NGE 2+00 D/S

LEGEND

— 1981 BASELINE
○ 16 APRIL 1984
□ 14 MAY 1984
△ 11 JUNE 1984

Smithville Dam
RP-3-1691
APPENDIX A PLATE 17 July 1984



ALINEMEN

R

LEGEND

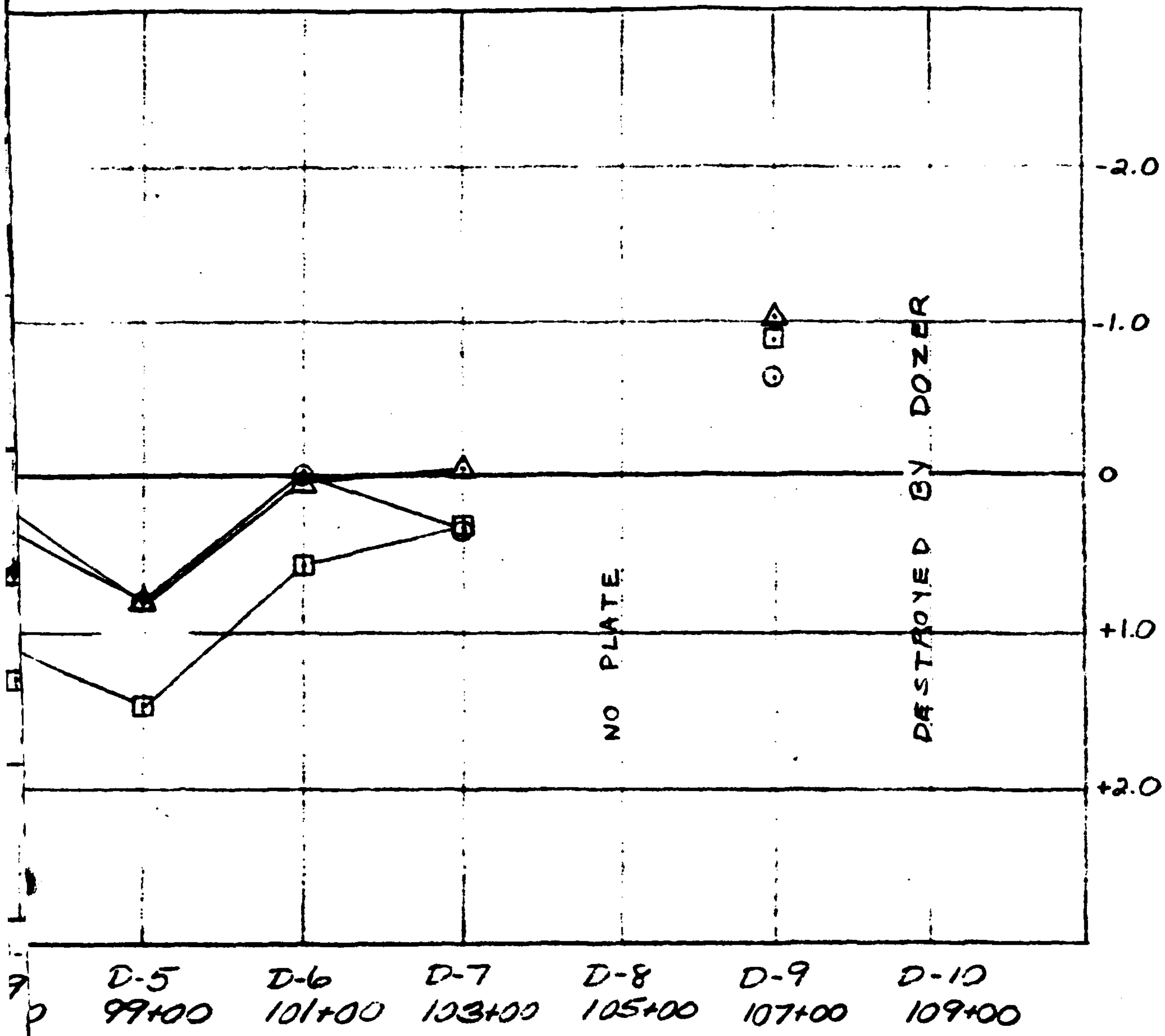
— 1980 BASELINE

○ 16 APRIL 1984

□ 14 MAY 1984

△ 4 JUNE 1984

15-1-7



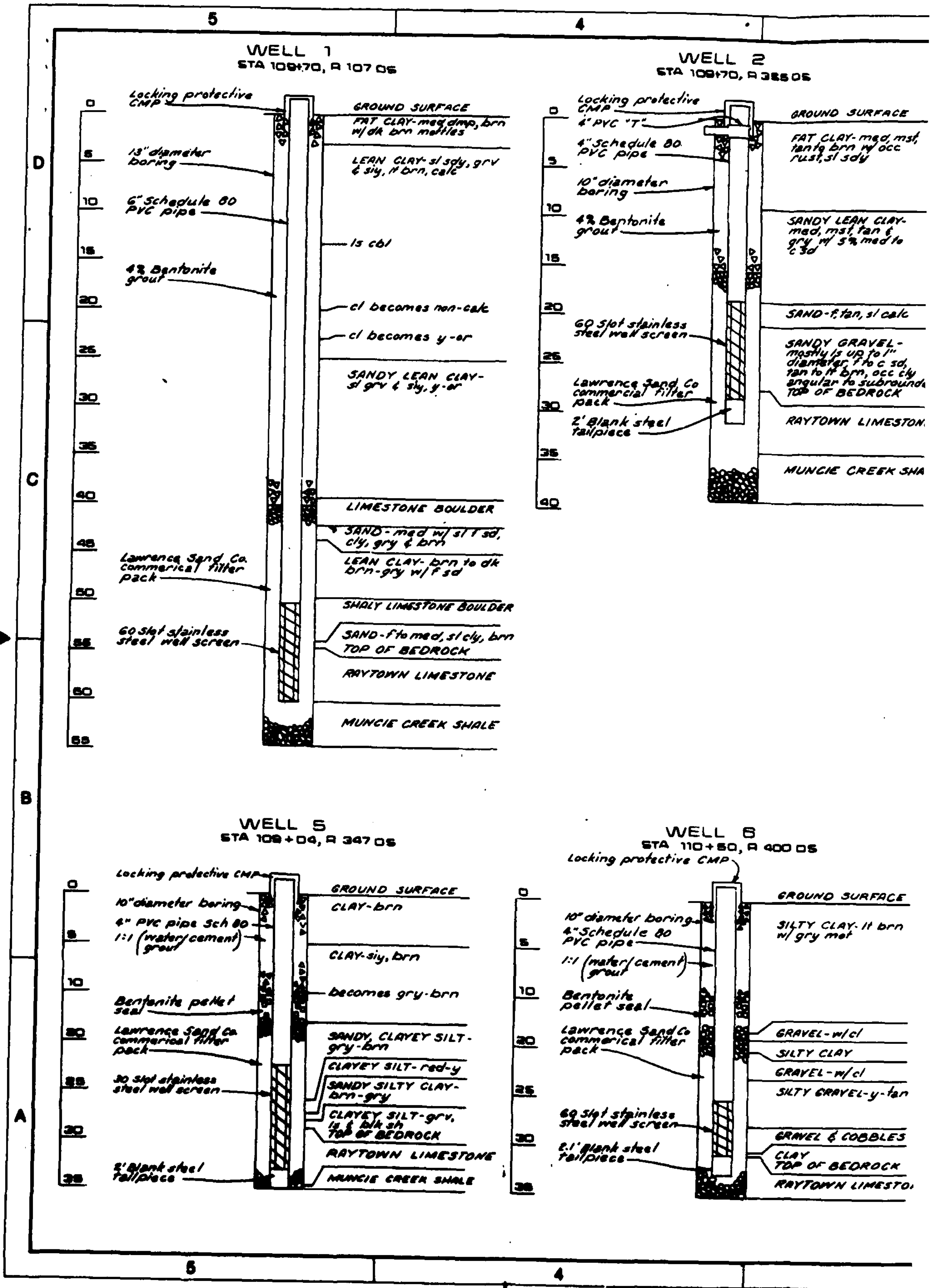
MENT MONUMENT LINE "D"
RANGE 4+00D/S

SMITHVILLE DAM
LEFT ABUTMENT

ALINEMENT LINE "D"
TYPICAL DATA SHEET

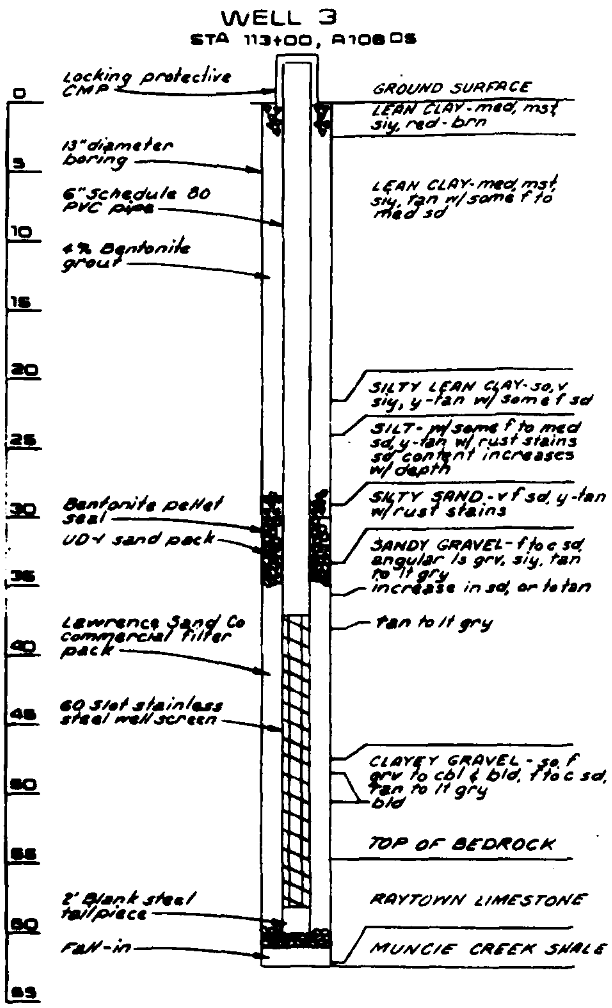
APPENDIX A RP-3-1692
PLATE 18 JULY 1984

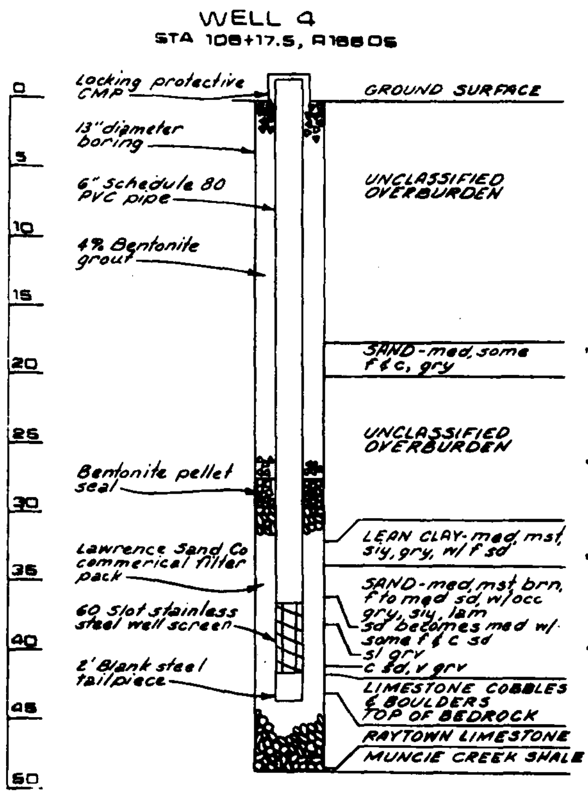
2



5
4
D
C
B
A
WELL 1
STA 109+70, R 107 OS
Locking protective CMP
13" diameter boring
6" Schedule 80 PVC pipe
4% Bentonite grout
Lawrence Sand Co. commerical filter pack
60 Slot stainless steel well screen
GROUND SURFACE
FAT CLAY-med dmp, brn w/dk brn mottles
LEAN CLAY-sl sdy, grv & sly, lt brn, calc
ls cbl
cl becomes non-calc
cl becomes y-or
SANDY LEAN CLAY-sl grv & sly, y-or
LIMESTONE BOULDER
SAND-med w/ sl f sd, cly, gry & brn
LEAN CLAY-brn to dk brn-gry w/ f sd
SHALY LIMESTONE BOULDER
SAND-f to med, sl cly, brn
TOP OF BEDROCK
RAYTOWN LIMESTONE
MUNCIE CREEK SHALE
WELL 2
STA 109+70, R 385 OS
Locking protective CMP
4" PVC "T"
4" Schedule 80 PVC pipe
10" diameter boring
4% Bentonite grout
60 Slot stainless steel well screen
Lawrence Sand Co commercial filter pack
2' Blank steel Tailpiece
GROUND SURFACE
FAT CLAY-med, mst, tan to brn w/ occ rust, sl sdy
SANDY LEAN CLAY-med, mst, tan & gry w/ 5% med to c sd
SAND-f, tan, sl calc
SANDY GRAVEL-mostly ls up to 1" diameter, f to c sd, tan to lt brn, occ cly angular to subrounde
TOP OF BEDROCK
RAYTOWN LIMESTON
MUNCIE CREEK SHA
WELL 5
STA 109+04, R 347 OS
Locking protective CMP
10" diameter boring
4" PVC pipe Sch 80
1:1 (water/cement) grout
Bentonite pellet seal
Lawrence Sand Co commerical filter pack
30 Slot stainless steel well screen
2' Blank steel Tailpiece
GROUND SURFACE
CLAY-brn
CLAY-sly, brn
becomes gry-brn
SANDY, CLAYEY SILT-gry-brn
CLAYEY SILT-red-y
SANDY SILTY CLAY-brn-gry
CLAYEY SILT-grv, ls & blk sh
TOP OF BEDROCK
RAYTOWN LIMESTONE
MUNCIE CREEK SHALE
WELL 6
STA 110+50, R 400 OS
Locking protective CMP
10" diameter boring
4" Schedule 80 PVC pipe
1:1 (water/cement) grout
Bentonite pellet seal
Lawrence Sand Co commerical filter pack
60 Slot stainless steel well screen
2.1' Blank steel Tailpiece
GROUND SURFACE
SILTY CLAY-lt brn w/ gry mot
GRAVEL-w/cl
SILTY CLAY
GRAVEL-w/cl
SILTY GRAVEL-y-tan
GRAVEL & COBBLES
CLAY
TOP OF BEDROCK
RAYTOWN LIMESTO

15-1-7

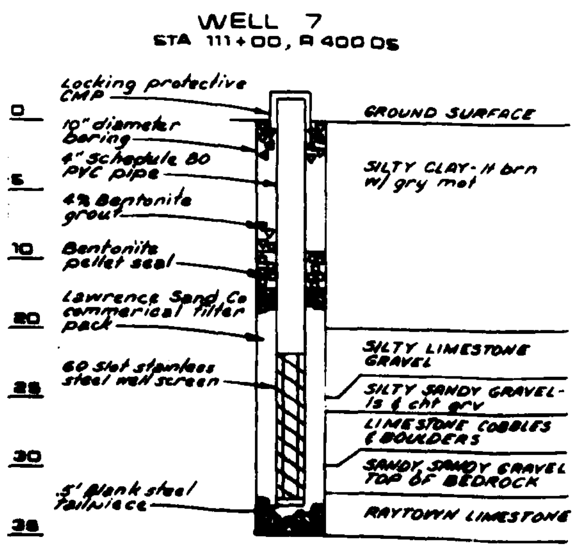




PIEZOMETER SCHEDULE

Piezometer Number	Dam Station	Range	①Surface Elevation	Tip Elev	Material adjacent to Slotted tip
P-106-4	106+02	15 DS	895* top dam	808.0	f to med siy, cly sd
P-106-5	106+00	150 DS	857.4	807.2	cl w/ grv
P-106-6	105+55	240 DS	846.4	802.8	sdy, siy cl
P-107-3	107+00	136 DS	859.2	808.7	sdy lean cl, w/ os'sd lens
P-108-1	108+10	234.5 DS	847.4	804.4	Muncie Creek Shale
P-108-2	107+60	150 DS	857.7	810.2	sdy cl
P-109-1	109+00	240 DS	846.9	809.5	slty cl
P-109-2	109+10	240 DS	846.8	789.8	Chanute Shale
P-110-3	110+00	20 US	853.4	813.4	cly, sdy grv
P-110-4	110+00	320 DS	839.1	813.3	sdy lean cl w/ grv
P-110-6	110+00	348 DS	838.1	808.1	ls grv & cbl w/ sdy cl
P-110-7	110+00	423 DS	832.5	809.5	cl & grv
P-110-8	110+00	109 DS	862.0	811.0	cly, sdy grv w/ cbl
P-110-9	110+00	240 DS	846.8	808.6	frac ls bld
P-110-10	109+47	138.5 DS	858.4*	802.6	Raytown Limestone
P-112-1	111+60	150 DS	857.9	813.6	sdy, siy grv w/ cl & cbl to bld
P-112-2	112+00	250 DS	850.5*	802.5	Raytown Limestone
P-112-3	111+80	300 DS	845.88*	[illegible]	ls & cht grv w/ cly si
P-113-1	113+00	177 DS	857.6	818.6	sdy, siy grv
P-114-1	114+00	100 DS	865.6	817.6	sdy cl
P-118-1	118+00	100 DS	868.5	802.0	grvly, cly sd w/ cbl & bld
P-119-1	119+00	75 DS	877.0*	824.0	sd grv & cbl in cl mtx
P-106-7	105+50	137 DS	868.69	786.7	Chanute Shale

① Unless noted by "*" elevations are from survey taken 5-84



SYM.	DESCRIPTION	DATE	APPR.
	REVISIONS		

PLATTE RIVER BASIN

SMITHVILLE RESERVOIR

LITTLE PLATTE RIVER

WELL CONSTRUCTION DETAILS AND PIEZOMETER SCHEDULE

Scale: as shown

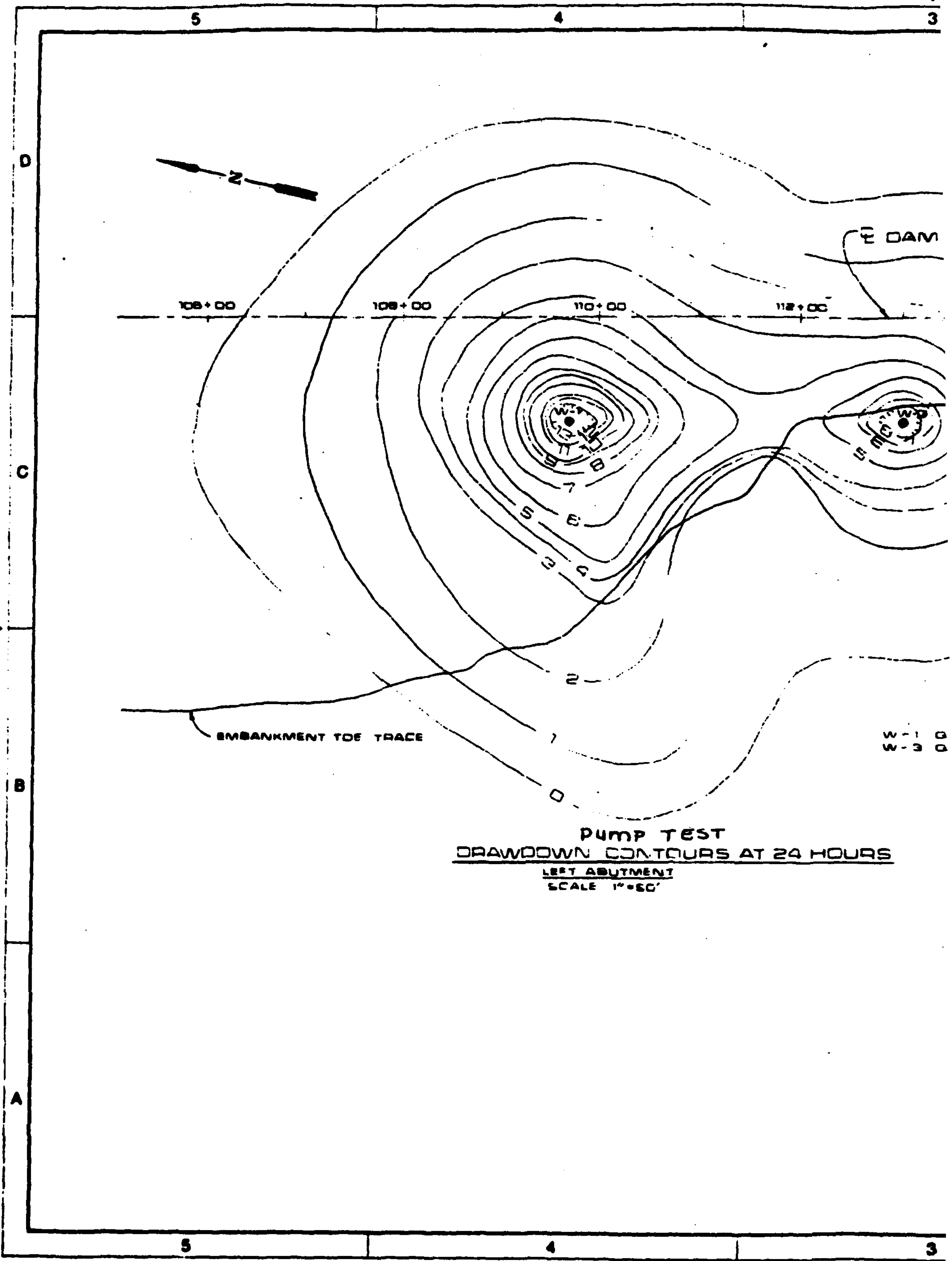
CORPS OF ENGINEERS KANSAS CITY DISTRICT

U. S. ARMY

Submitted: Recommended: Approved:

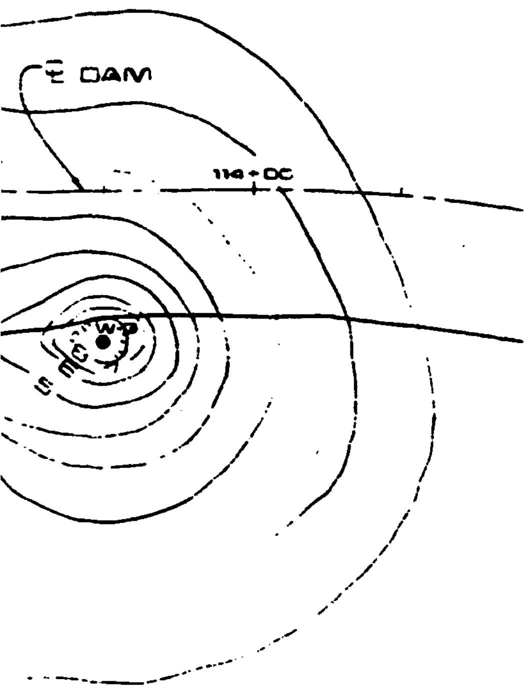
RP3-1693

APPENDIX A PLATE NO. 89



PUMP TEST
DRAWDOWN CONTOURS AT 24 HOURS
LEFT ABUTMENT
SCALE 1"=50'

15-1-7



W-1 Q = 7.5 GPM
W-3 Q = 17 GPM

OURS

SYM.	DESCRIPTION	DATE	APP'D.

REVISIONS

PLATTE RIVER BASIN

SMITHVILLE RESERVOIR

LITTLE PLATTE RIVER

Pump Test
DRAWDOWN CONTOURS

Scale: as shown

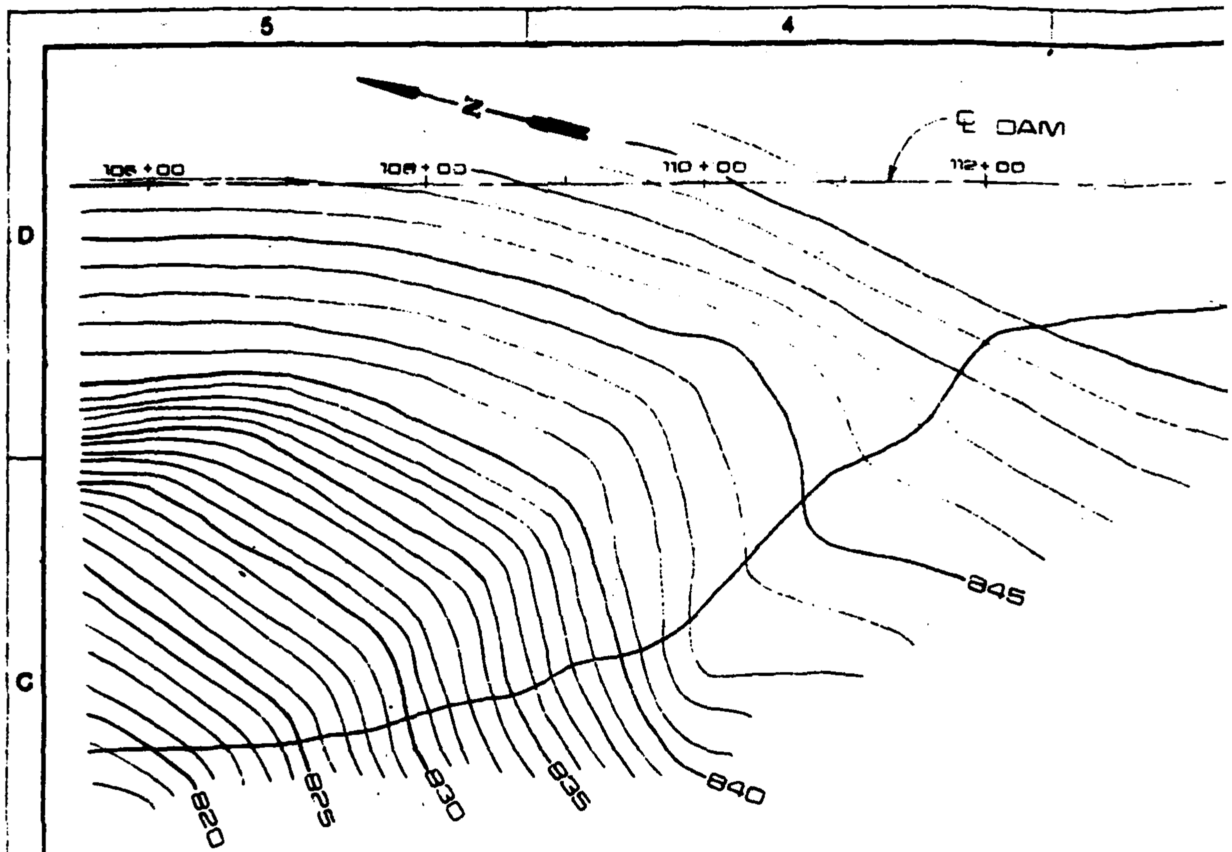
U. S. ARMY

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

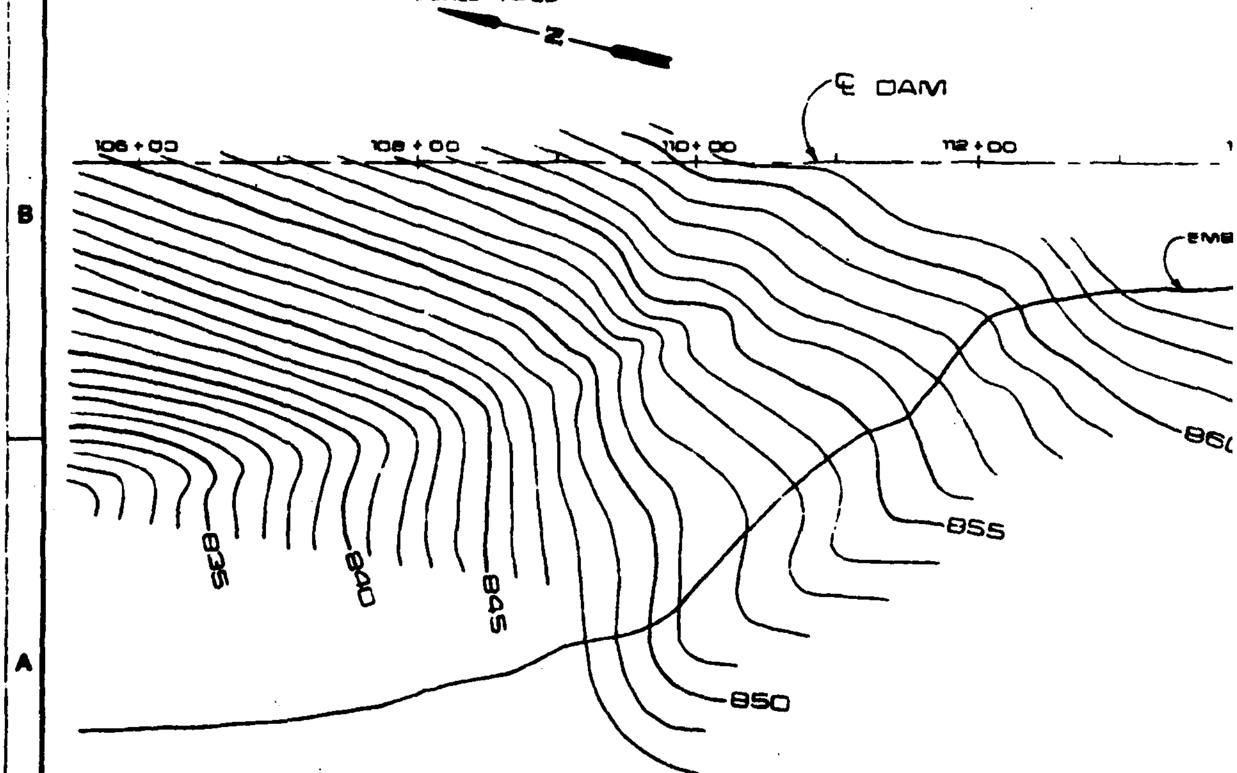
RP-3-1692

APPENDIX A PLATE NO. 20

2

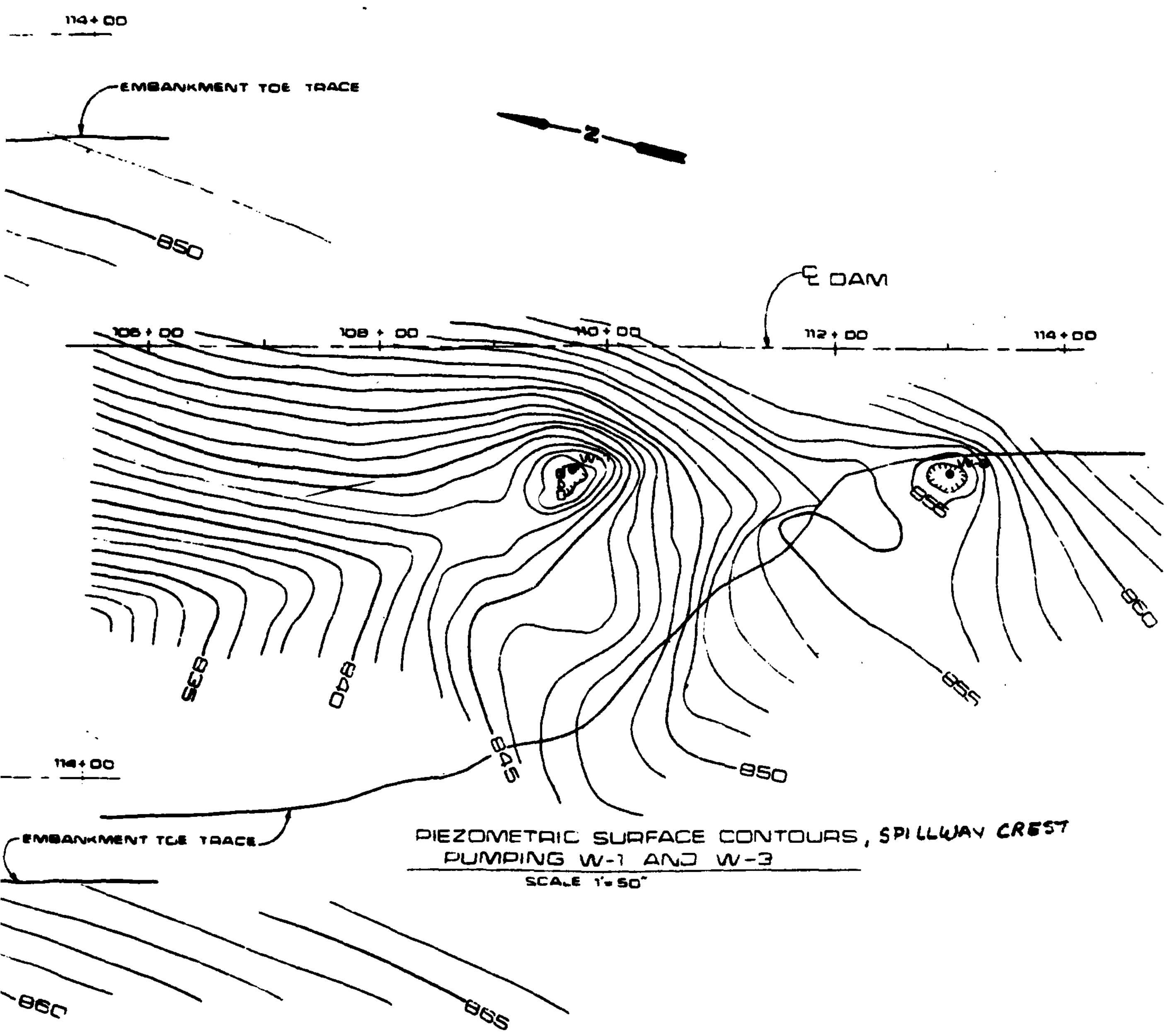


PIEZOMETRIC SURFACE CONTOURS
AT MULTIPURPOSE POOL, El 864.0
SCALE 1"=50'



PIEZOMETRIC SURFACE CONTOURS PROJECTED TO POOL
AT SPILLWAY CREST, El 880.2
SCALE 1"=50'

15-1-7



PIEZOMETRIC SURFACE CONTOURS, SPILLWAY CREST
PUMPING W-1 AND W-3
SCALE 1"=50'

SYM.	DESCRIPTION	DATE	APP'D.
	REVISIONS		

LITTLE PLATTE RIVER, MISSOURI

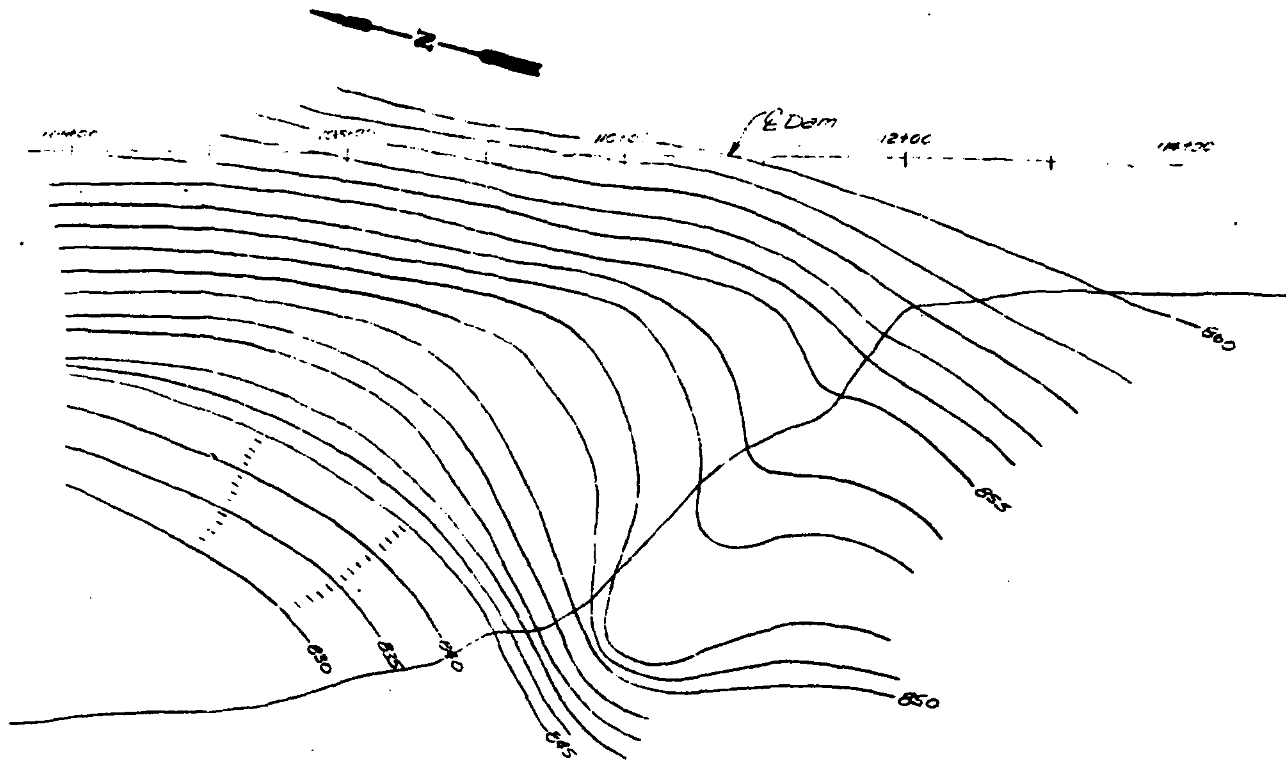
SMITHVILLE LAKE

PIEZOMETRIC CONTOURS
INTERIM ANALYSIS

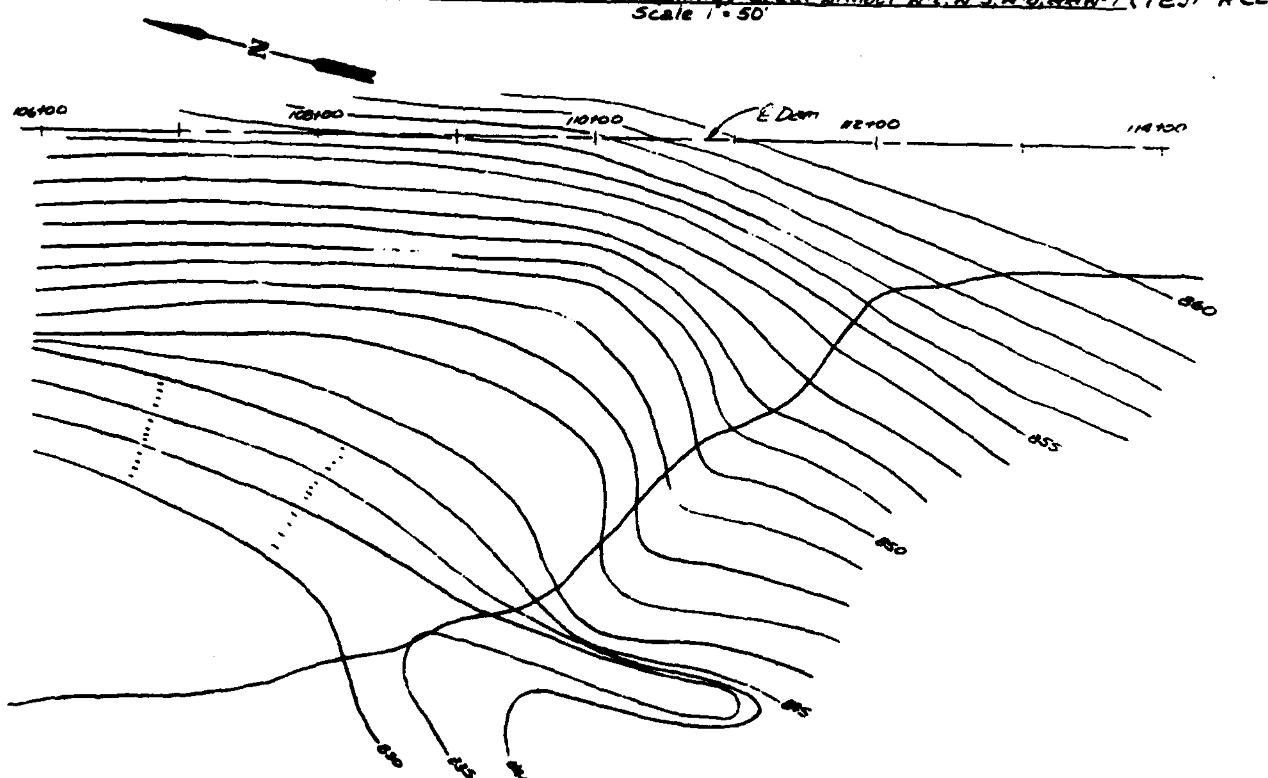
Dwg. No. — Scale: as shown
CORPS OF ENGINEERS, KANSAS CITY DISTRICT — U. S. ARMY

RP-3-1695

APPENDIX A PLATE NO. 21



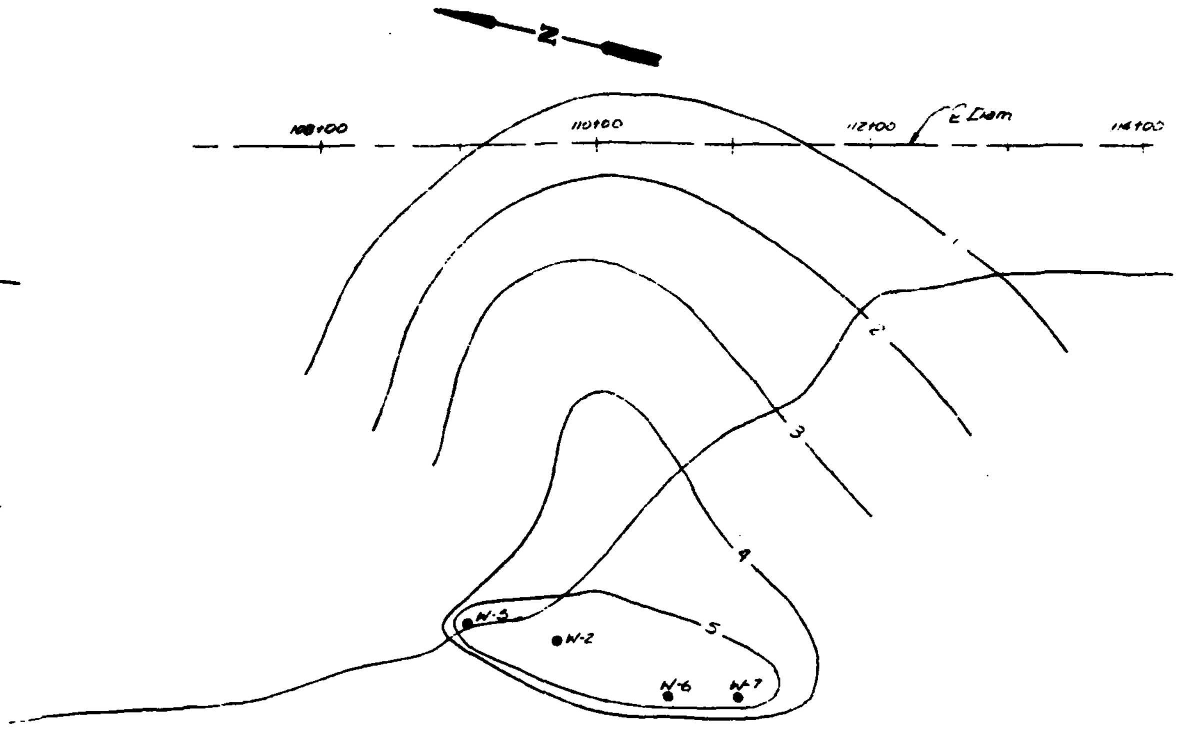
Piezometric Contours - Pool at Spillway Crest without W-2, W-5, W-6, and W-7 (TEST REL
Scale 1" = 50'



Piezometric Contours - Pool at Spillway Crest with W-2, W-5, W-6, and
Scale 1" = 50'

5-1-7

3 | 2 | 1



RELIEF WELLS)

Drawdown Contours W-2, W-5, W-6, W-7 (TEST RELIEF WELLS)
Scale 1"=50'.

W-6 and W-2 (TEST RELIEF WELLS)

Symbol	Descriptions	Date	Approved

Revisions

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by:
Drawn by:
Checked by:
Submitted by:

US Army Corps of Engineers

SMITHVILLE LAKE

PIEZOMETRIC CONTOURS

FINAL ANALYSIS

Scale:
Date:
Dwg. No.:
Sheet number:

RP-3-1696

3 | 2 | 1

APPENDIX A PLATE NO. 22

DIRECT SHEAR & RESIDUAL SHEAR RESULTS

SYMBOL	SAMPLE NUMBER	GEOLOGIC MEMBER	NORMAL STRESS	DRAINED SHEAR STRENGTH	
				PEAK	RESIDUAL
			σ_n	tan Ø	tan $Ø_r$
1	J-108-1	RaLs. -THK SH SEAM	6.0	.328*	.147*
2	C-525	RaLs. - SH SEAM @ 40'	6.0	.561	***
3	C-525	RaLs. -THK SH SEAM W/ SLICKENSIDES	6.0	.416	.196
4	C-525	RaLs - THK SH SEAM W/O SLICKS	6.0	.469	.191
5	C-525	RaLs / Mc SH CONTACT	6.0	.639	.387***
6	C-525	CH SH -Lo ANGLE SOFT ZONE	6.0	.499	.226***
7	C-526	RaLs - THK SH SEAM	6.0	.230**	.13 **
8	C-526	RaLs -THK SH SEAM	6.0	.296**	.13 **
9	C-526	Mc SH	6.0	.423	.213**
10	C-527	RaLs - THN SH SEAM	6.0	.59	.23***
11	C-528	RaLs - THK SH SEAM	6.0	.289	.129***
12	C-529	Mc SH	6.0	.559	.215***
13	C-532	RaLs - THK SH SEAM	4.0	.313	

.202

*REMOLDED SPECIMEN

**PRECUT SPECIMEN

***DID NOT DEVELOP RESIDUAL CONDITION

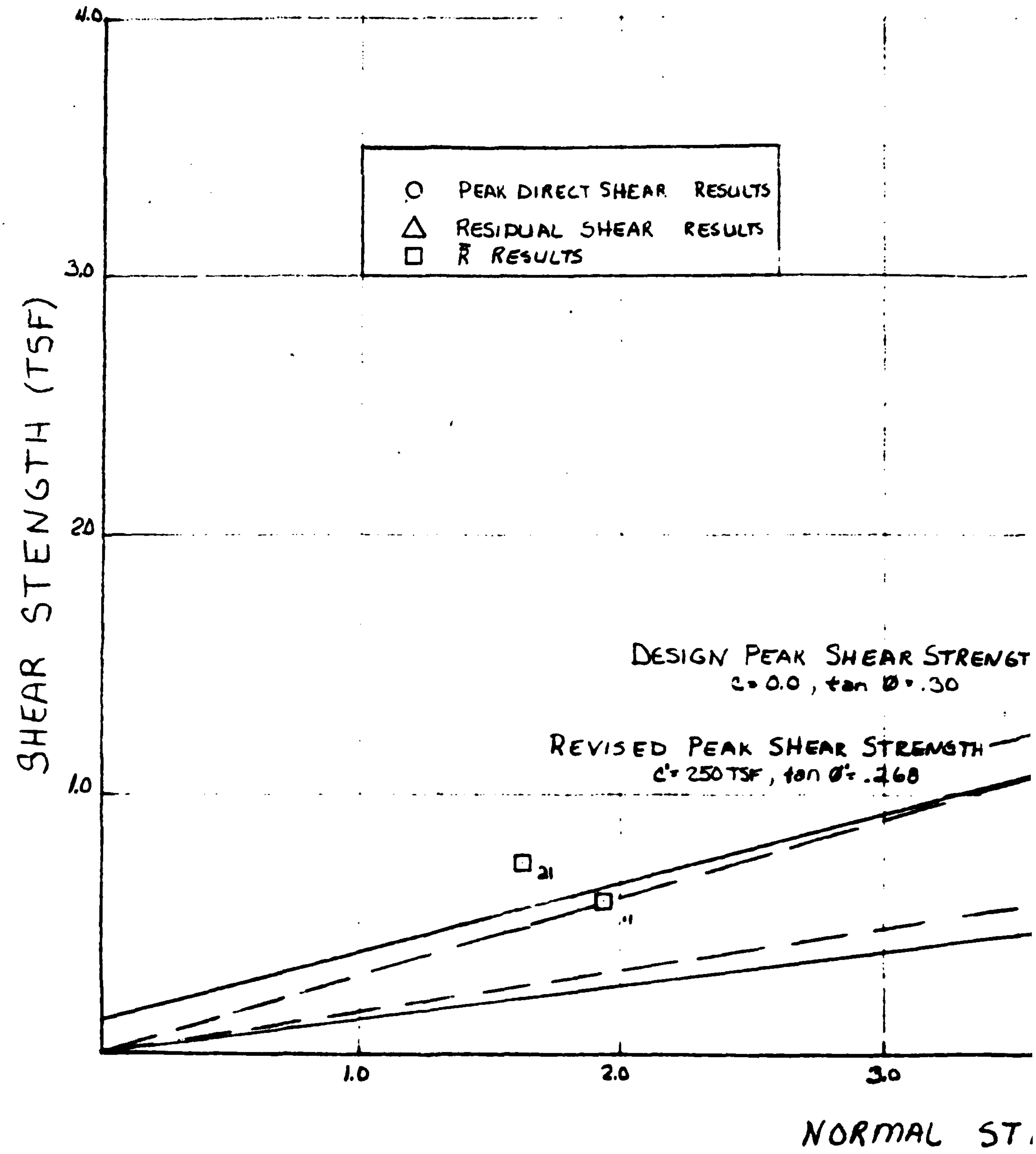
TRIAXIAL "$\bar{R}$" RESULTS

SYMBOL	SAMPLE	σ_{3c}	τ_{ff}	σ_{ff}
$\bar{R}_{11}$	C-527	2	.59	1.94
$\bar{R}_{12}$	C-527	6	1.40	5.22
$\bar{R}_{21}$	C-533	2	.73	1.62
$\bar{R}_{22}$	C-533	6	1.81	4.96

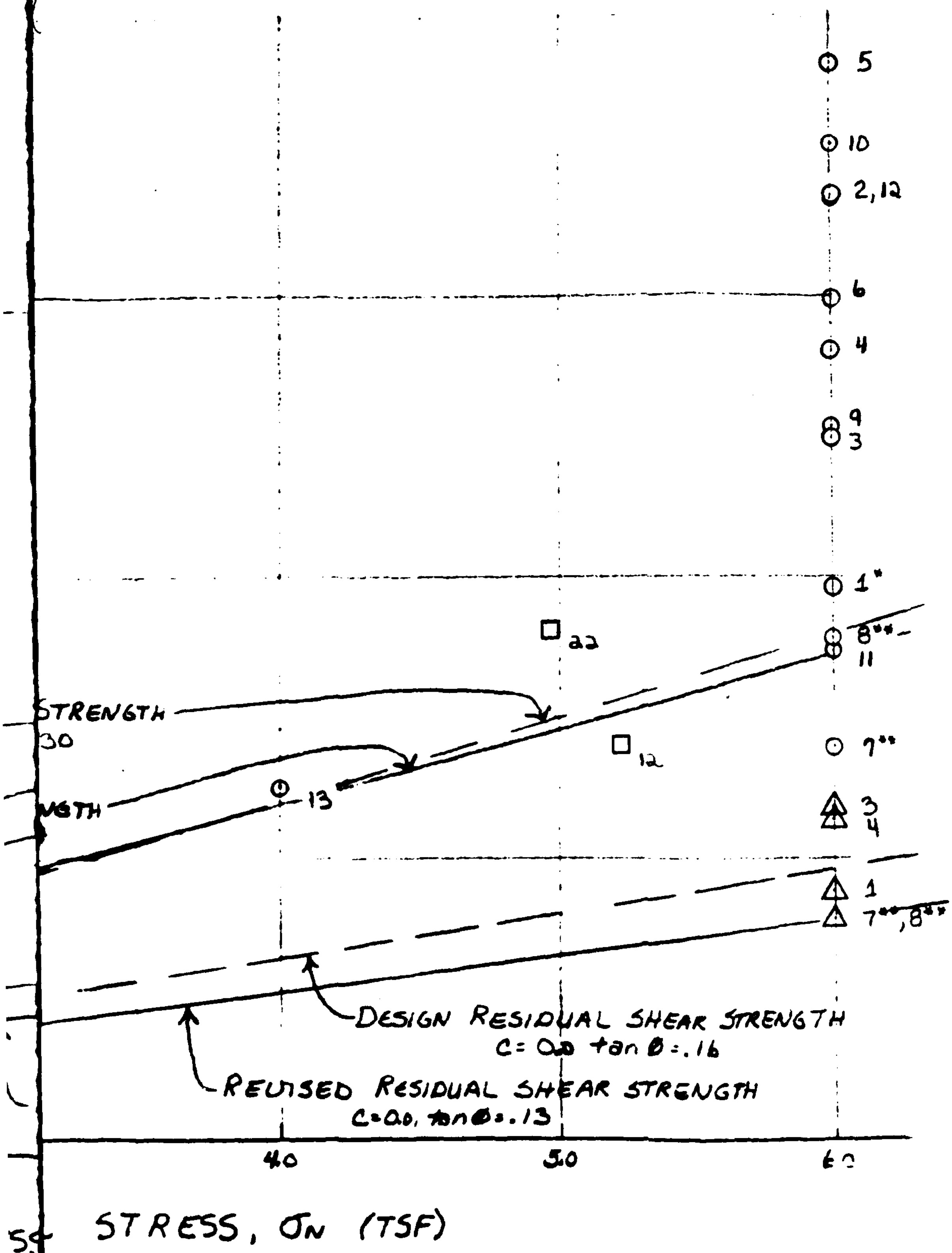
SMITVILLE DAM
LEFT ABUTMENT

SUMMARY OF SHEAR STENGTH RESULTS

APPENDIX A — RP-3-1697
PLATE 23 — JULY 1984


4.0
3.0
20
1.0
SHEAR STENGTH (TSF)
O PEAK DIRECT SHEAR RESULTS
△ RESIDUAL SHEAR RESULTS
□ R̄ RESULTS
DESIGN PEAK SHEAR STRENGT
c = 0.0 , tan Ø = .30
REVISED PEAK SHEAR STRENGTH
c' = 250 TSF , tan Ø' = .268
21
11
1.0
2.0
3.0
NORMAL ST

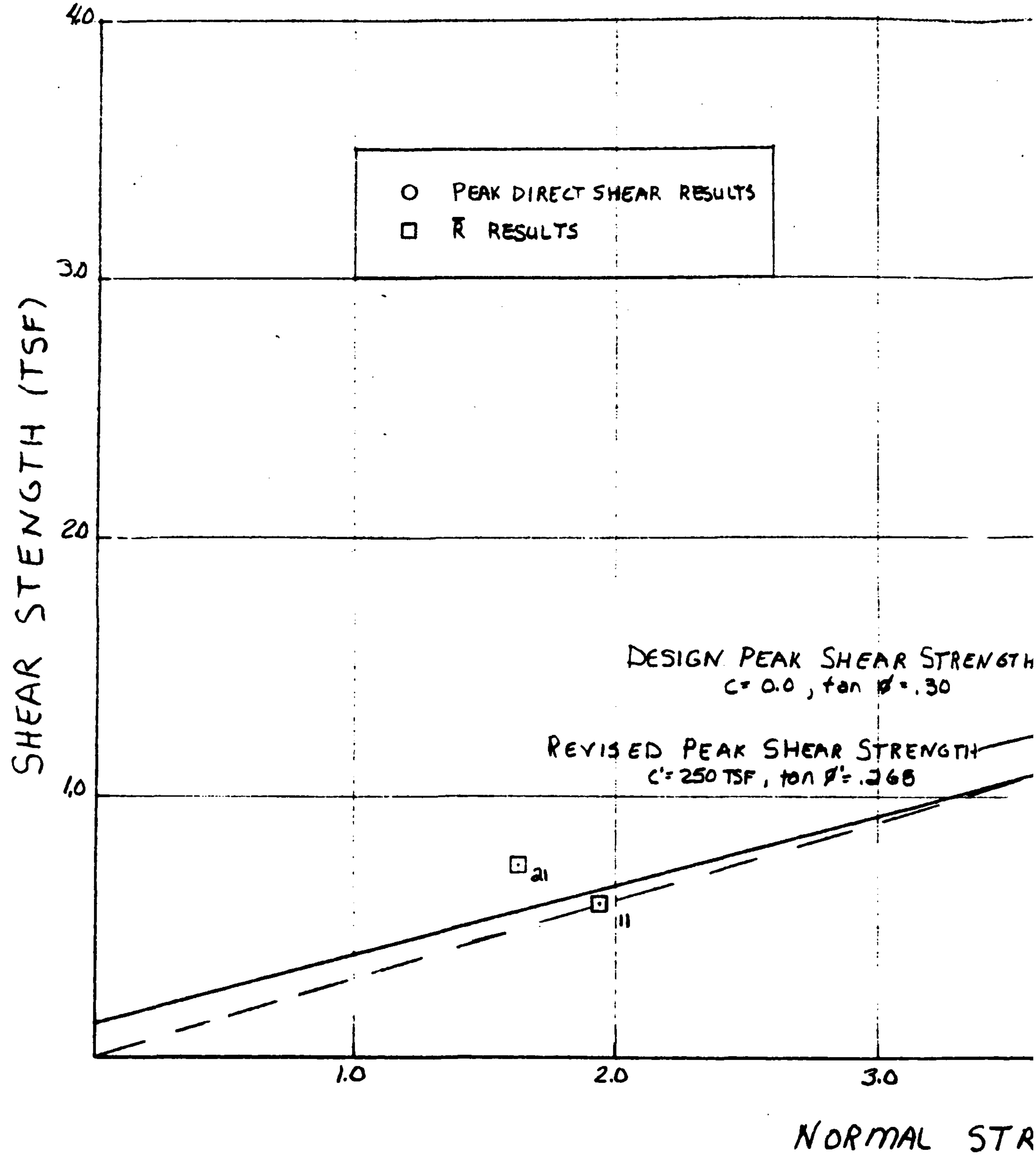
15-1-7



SMITHVILLE DAM:
LEFT ABUTMENT

PLOT OF SHEAR STRENGTH RESULTS

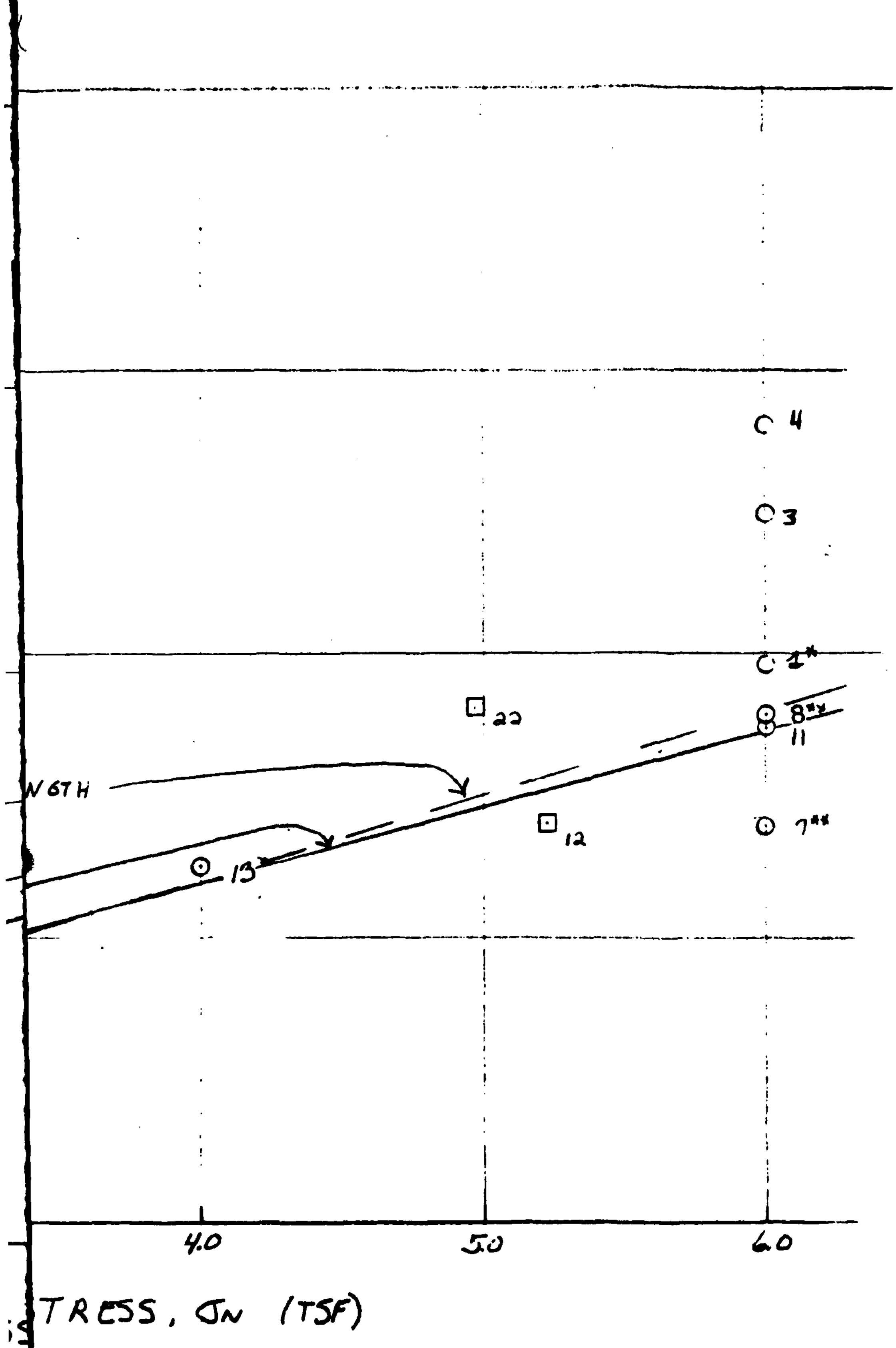
RP-3-1698
APPENDIX A
PLATE 24 JULY 1984



PEAK DIRECT SHEAR RESULTS
R̄ RESULTS
4.0
3.0
2.0
1.0
SHEAR STENGTH (TSF)
DESIGN PEAK SHEAR STRENGTH
C= 0.0 , tan Ø = .30
REVISED PEAK SHEAR STRENGTH
C'= 250 TSF , tan Ø'= .268
21
11
1.0
2.0
3.0
NORMAL STR

15-1-7

1



SMITHVILLE DAM

LEFT ABUTMENT

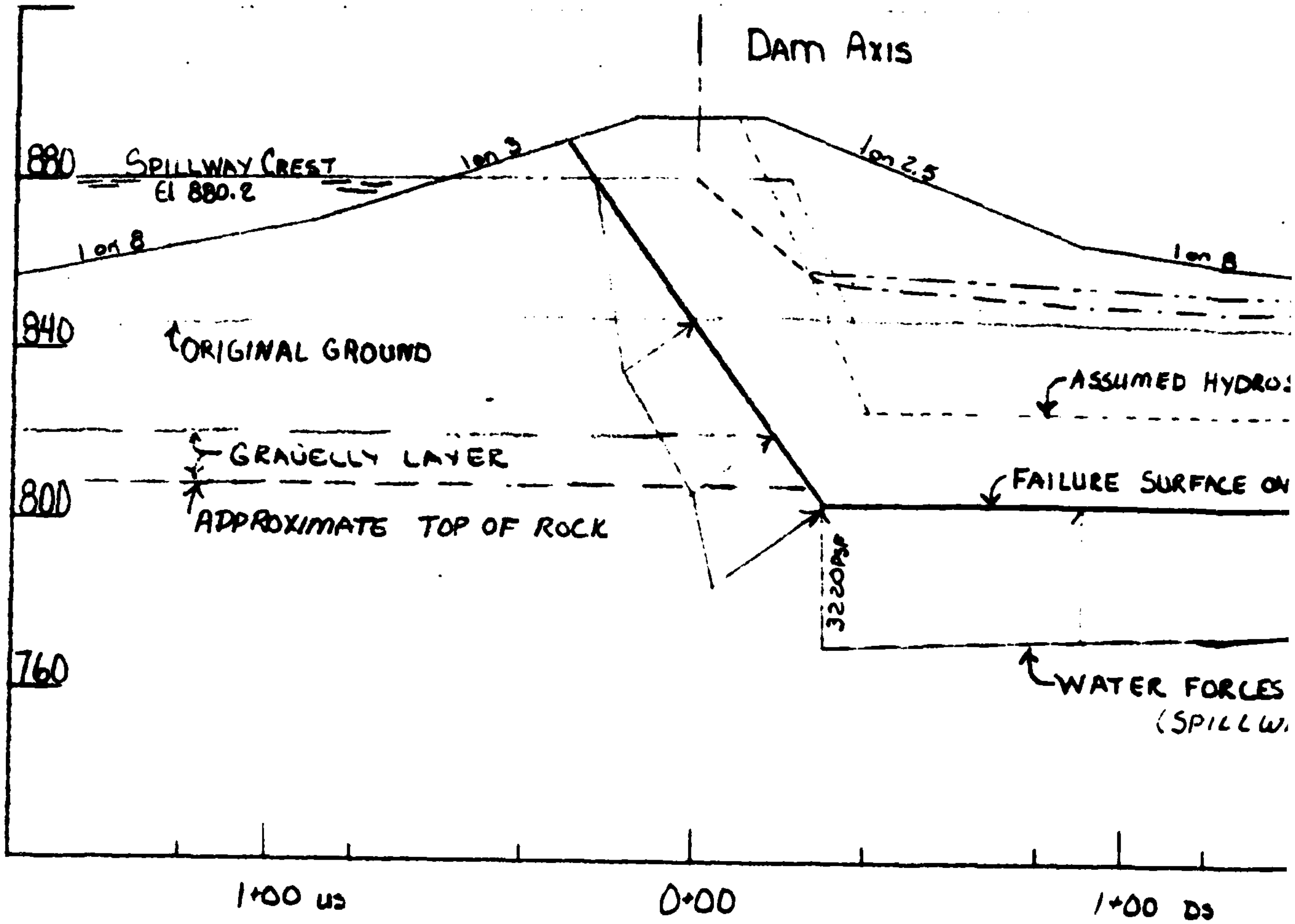
PLOT OF SHEAR STRENGTH RESULTS

RAYTOWN SHALE SEAM

RP-3-1699

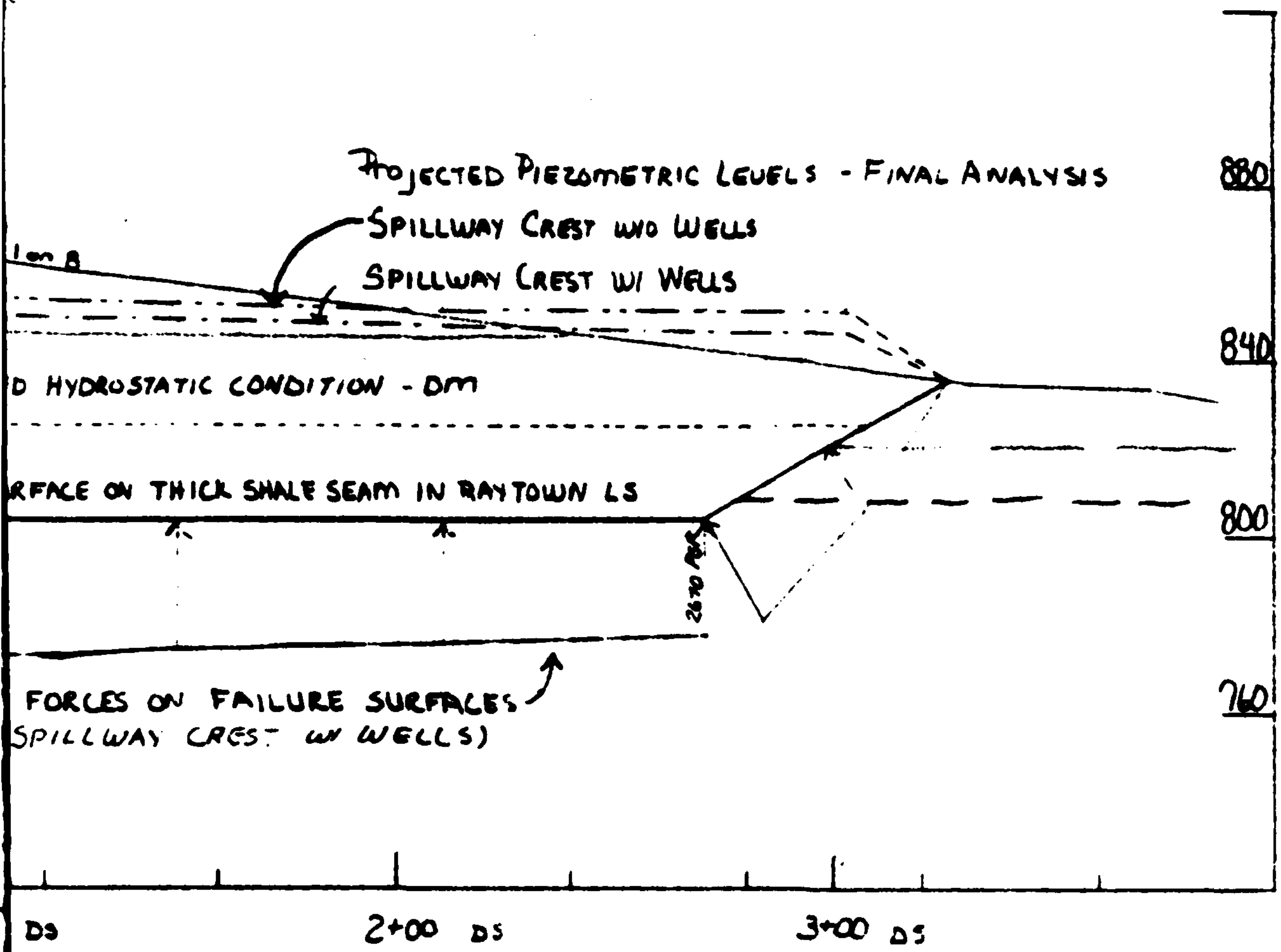
APPENDIX A

PLATE 25 JULY 1984



STABILITY SECTION STA 110+

15-1-7



110+00

SMITHVILLE DAM
LEFT ABUTMENT

STABILITY SECTION
STA 110+00

APPENDIX A RP-3-1700
PLATE 26 JULY 1984

2

	SHEAR STRENGTH PARAMETERS					
ANALYSIS	DM ; PRELIM		INTERIM		FINAL	
PEAK STRENGTH	c (PSF)	tan ø	c(PSF)	tan ø	c'(PSF)	[illegible]
EMBANKMENT*	0/300	.45/.325	0	.45	0	[illegible]
DUBN - FINE GRAINED*	0/200	.45/.325	0	.45	0	[illegible]
DUBN - GRAVELLY	200	.325	0	.577	0	[illegible]
JOINT IN RAYTOWN	~	~	0	.577	0	[illegible]
RAYTOWN LS SH SEAM	0	.30	160	.264	250	[illegible]
RESIDUAL STRENGTH						
RAYTOWN LS SH SEAM	0	.16	~	~	~	[illegible]
STR @ 0.5% STRAIN						
DY BN - PASSIVE WEDGE	~	~	40	.13	40	[illegible]

* DM AND PRELIMINARY ANALYSES USED a S,(R+S)/2 ENVELOPE

INTERIM AND FINAL ANALYSES USED A S ENVELOPE

15-1-7

S	
AL	
	tan Ø
	.45
	.45
	.577
	6
	.268
	~
	.13

PHYSICAL SOIL PROPERTIES		
MATERIAL	UNIT WEIGHT LBS/FT³	
	SAT	MOIST
EMBANKMENT	125	120
OVERBURDEN	120	115
BEDROCK	140	140

SMITHVILLE DAM
LEFT ABUTMENT

DESIGN PARAMETERS

APPENDIX A RP-3-1701
PLATE 27 JULY 1984

DM ANALYSIS

POOL CONDITION[1]	STRENGTH APPROACH[2]	METHOD: HAND WEDGE[3]
SPILLWAY CREST	1	1.64
SPILLWAY CREST	2	1,08

PRELIMINARY ANALYSIS

POOL CONDITION[1]	STRENGTH APPROACH[2]	METHOD		
		HAND WEDGE[3]	SLOPE8R[4]	
			F	θ
FULL POOL	1	1.22	1.42	8.0
FULL POOL	2	0.92	1.00	6.9

INTERIM ANALYSIS*

POOL CONDITION[1]	STRENGTH APPROACH[2]	METHOD		
		HAND WEDGE[3]	SLOPE8R[4]	
			F	θ
MULTIPURPOSE	3	1.40	1.66	8.5°
SPILLWAY CREST	3	1.20	1.41	8.0°
SPILLWAY CREST W/ PUMPED WELLS	3	1.32	1.55	8.9°

PO	
CON	
SPILL w/o R	
SPILL w/ FL	

[1] Pool Con

[2] STRENGT

[3] HANDW

[4] SLOPE

* SEE F

FINAL ANALYSIS

POOL CONDITION[1]	STRENGTH APPROACH[2]	METHOD		
		HAND WEDGE[3]	SLOPE8R[4]	
			F	θ
SPILLWAY CREST w/o RELIEF WELLS	3	1.25	1.47	8.7
SPILLWAY CREST w/ FLOWING TEST WELLS	3	1.30	1.53	8.7

[1]Pool Condition : HYDROSTATIC PRESSURE WAS ASSUMED IN EMBANKMENT CORRESPONDING TO THE GIVEN POOL CONDITION. IN THE DM analysis, DOWNSTREAM UPLIFT CORRESPONDED TO A SATURATION LINE AT El. 825 IN THE FOUNDATION. IN ALL OTHERS ANALYSES, DOWNSTREAM UPLIFT CORRESPONDED TO ACTUAL OR PROJECTED PIEZOMETRIC LEVEL IN THE PERMEABLE BASAL LAYER IN THE FDN.

[2]STRENGTH APPROACH:

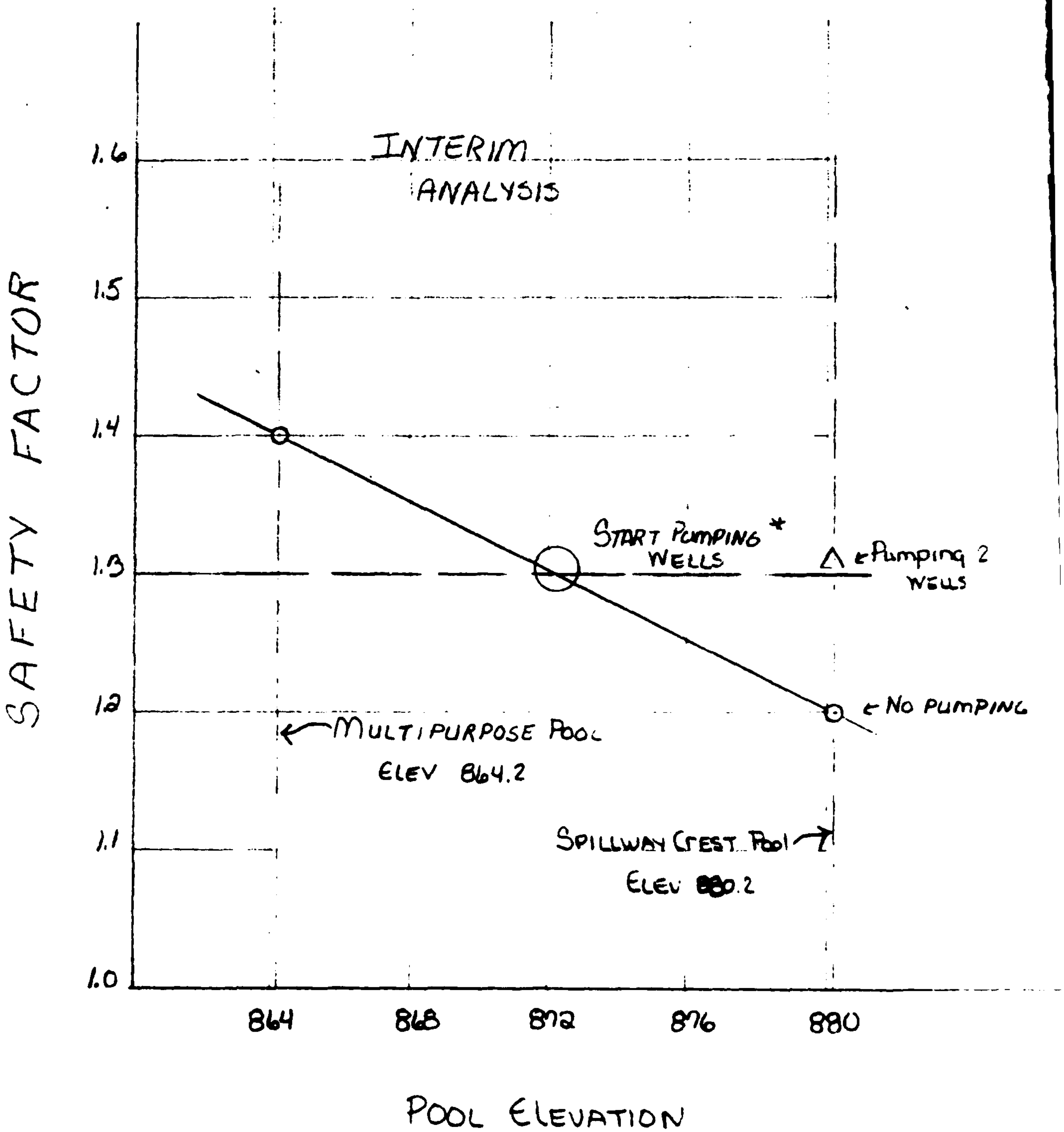
APPROACH 1 : PEAK STRENGTH USED ALONG ENTIRE SLIDE SURFACE
APPROACH 2 : PEAK STRENGTH USED IN ACTIVE AND PASSIVE WEDGE, RESIDUAL STRENGTH USED IN CENTRAL BLOCK
APPROACH 3 : PEAK STRENGTH USED IN ACTIVE WEDGE AND CENTRAL BLOCK; STRENGTH @ 0.5% STRAIN USED IN PASSIVE WEDGE

[3]HANDWEDGE : SAFETY FACTOR OBTAINED USED HAND WEDGE METHOD ACCORDING TO EM-1110-2-1906. (APRIL 1970)
IN DM ANALYSIS, $\epsilon_A = .08$, $\epsilon_P = 0.0$
IN OTHER ANALYSIS, $\epsilon_A = \epsilon_p = 0.0$

[4]SLOPE8R : F = FACTOR OF SAFETY
θ = SIDE FORCE INCLINATION

* SEE PLATE 29 FOR PLOT OF F VS Pool ELEV. FOR HAND WEDGE METHOD

SMITHVILLE DAM
LEFT ABUTMENT - STA 110+00
STEADY SEEPAGE STABILITY ANALYSIS SUMMARY
APPENDIX A RP-3-1702
PLATE 28 JULY 1984


INTERIM
ANALYSIS
SAFETY FACTOR
1.6
1.5
1.4
1.3
1.2
1.1
1.0
START PUMPING WELLS *
← Pumping 2 WELLS
← NO PUMPING
← MULTIPURPOSE POOL
ELEV 864.2
SPILLWAY CREST POOL
ELEV 880.2
864
868
872
876
880
POOL ELEVATION

15-1-7

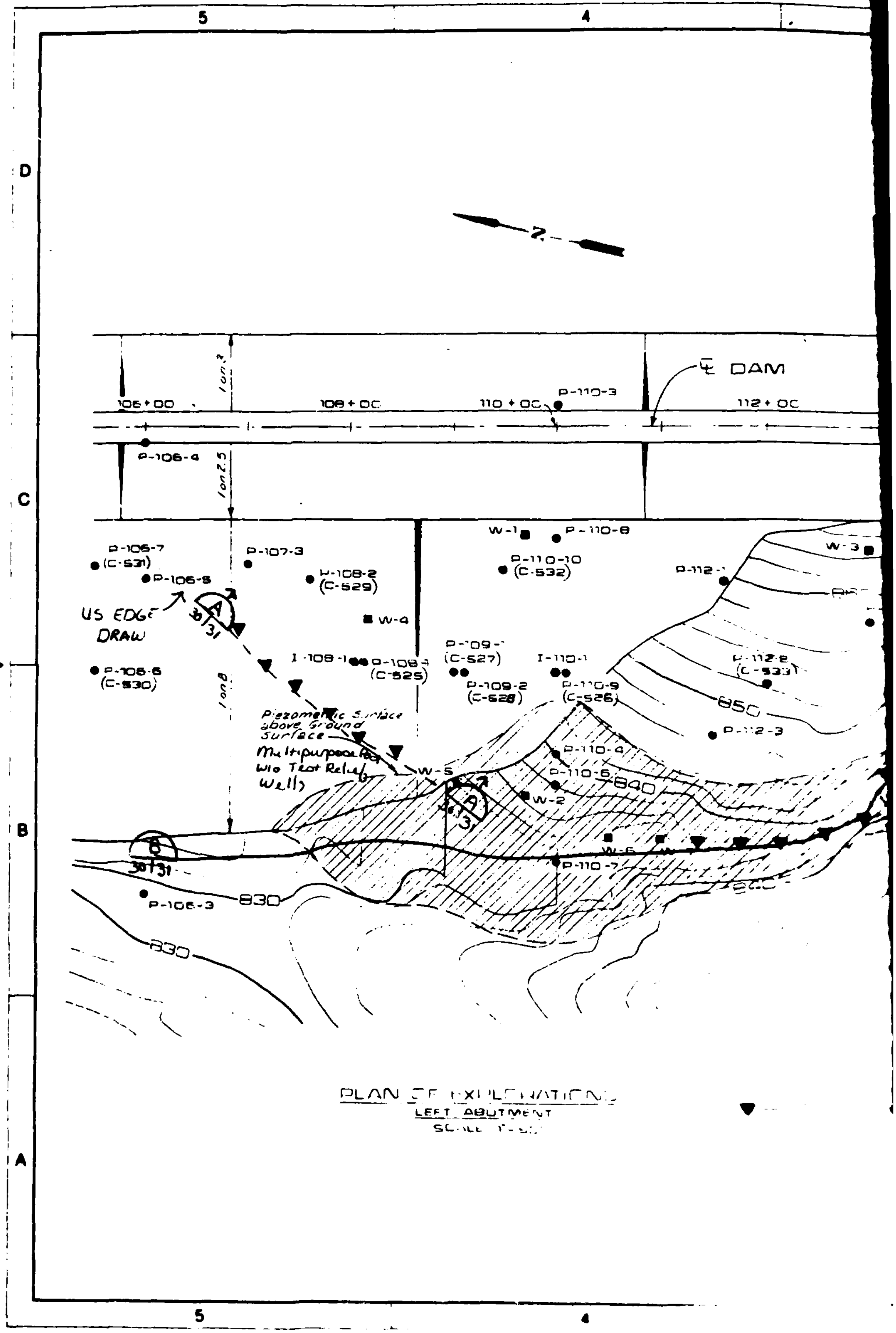
STABILITY STUDIES WERE CONDUCTED AT STATION 110+00 USING ACTUAL PIEZOMETRIC LEVELS AT MULTIPURPOSE POOL, PROJECTED PROJECTED LEVELS AT A SPILLWAY CREST, AND PREDICTED DRAWDOWN AT A SPILLWAY CREST POOL PUMPING 2 WELLS

* TO ASSURE A SAFETY FACTOR > 1.3, PUMPING WILL BE STARTED WHENEVER A POOL OF ELEV 872 OR HIGHER IS FORECAST.

SMITHVILLE DAM
LEFT ABUTMENT

SAFETY FACTOR VS POOL ELEV
INTERIM ANALYSIS

APPENDIX A   RP-3-1703
PLATE 29   JULY 1984

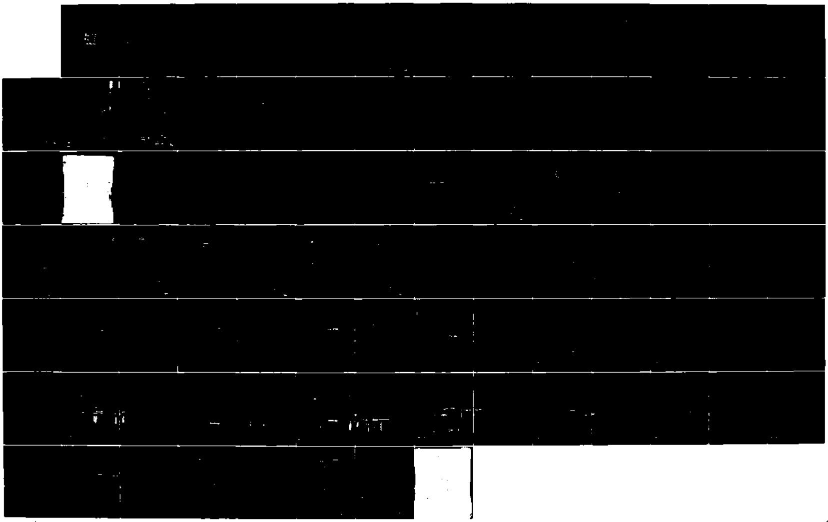


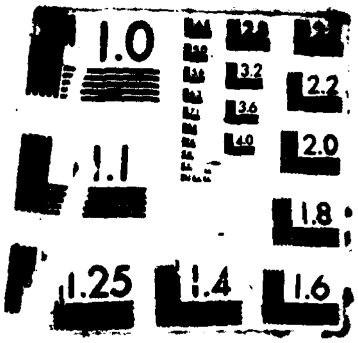
5
4
D
C
B
A
℄ DAM
106+00
108+00
110+00
112+00
P-110-3
P-106-4
1 on 3
1 on 2.5
1 on 8
W-1
P-110-8
P-110-10
(C-532)
W-3
P-112-1
P-106-7
(C-531)
P-107-3
P-106-5
P-108-2
(C-529)
US EDGE
DRAW
W-4
P-109-1
(C-527)
I-110-1
P-112-2
(C-533)
I-108-1
P-108-3
(C-525)
P-106-6
(C-530)
P-109-2
(C-528)
P-110-9
(C-526)
850
P-112-3
Piezometric Surface above Ground Surface
Multipurpose Pool w/o Test Relief Wells
W-5
P-110-4
P-110-6
840
W-2
W-6
P-110-7
P-106-3
830
LEFT ABUTMENT

15-1-7

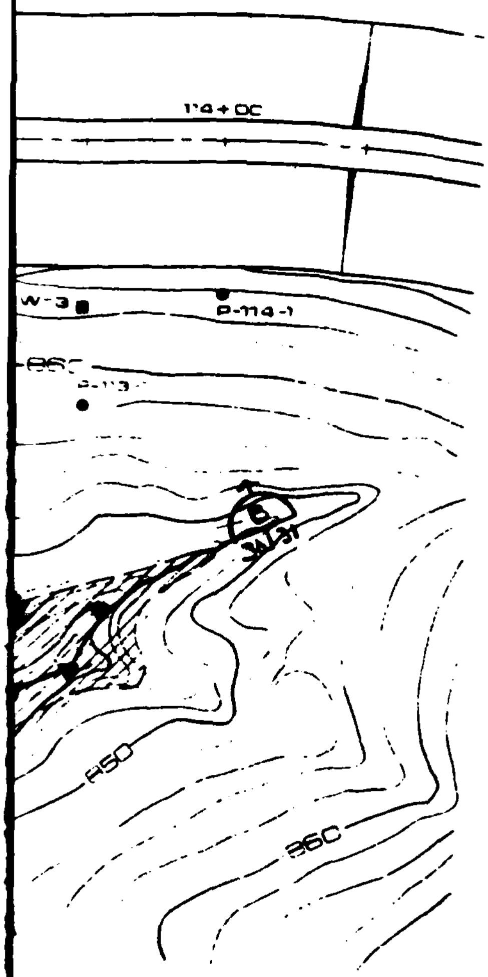
AD-A182 106 MULTIPLE-PURPOSE PROJECT: PLATTE RIVER BASIN LITTLE PLATTE RIVER MISSOURI..(U) CORPS OF ENGINEERS KANSAS CITY MO KANSAS CITY DISTRICT F C WALBERG ET AL. MAR 87 3/3

UNCLASSIFIED F/G 13/2 NL




1.0
2.2
3.2
3.6
4.0
2.0
1.1
1.8
1.25
1.4
1.6

3 | 2 | 1



...posed pressure
...ief wells

SYM.	DESCRIPTION	DATE	APP'D.

REVISIONS

LITTLE PLATTE RIVER, MISSOURI

SMITHVILLE LAKE

Proposed Pressure Relief Wells - Plan

Dwg. No.

Scale: as shown

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

U. S. ARMY

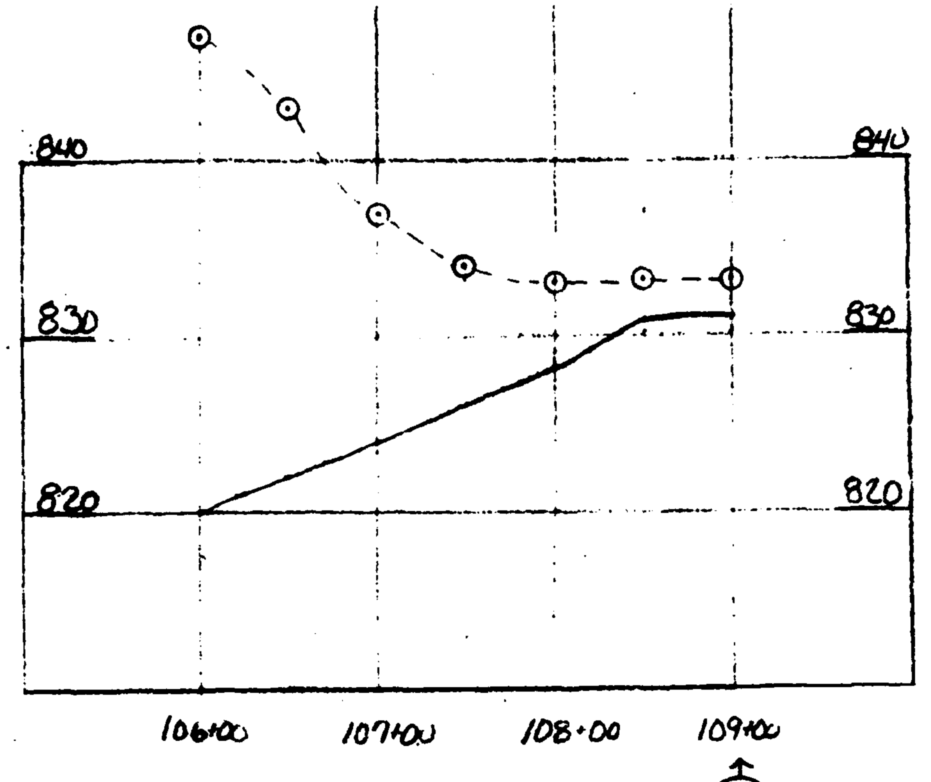
Submitted: Recommended: Approved:

RP-3-1704

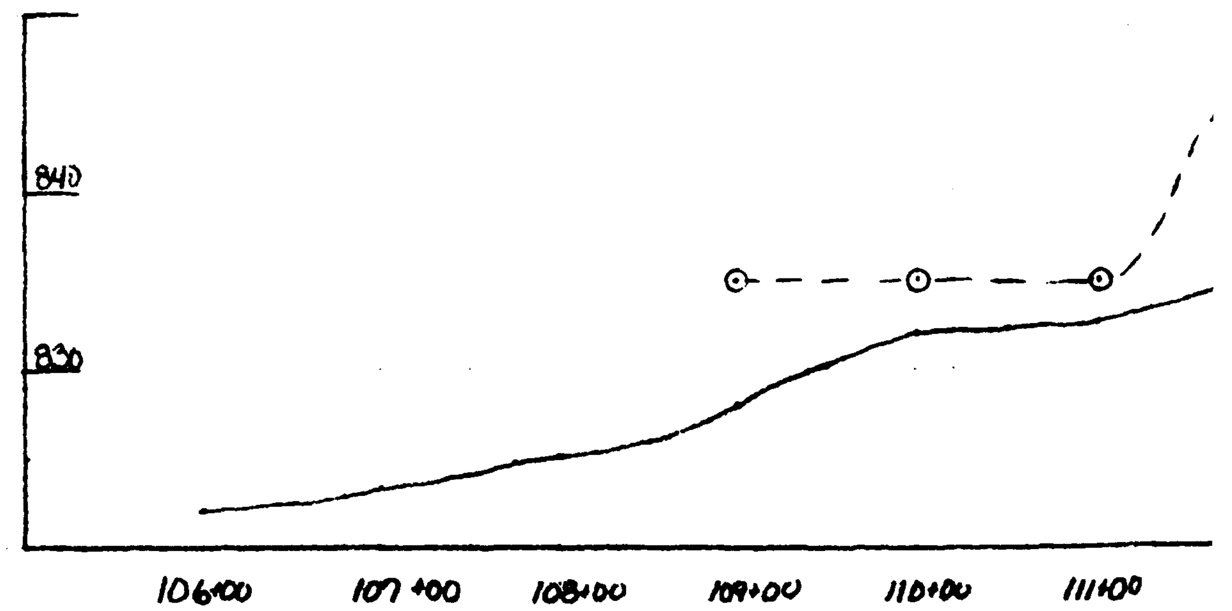
3 | 2 | APPENDIX A PLATE NO. 30

1

2



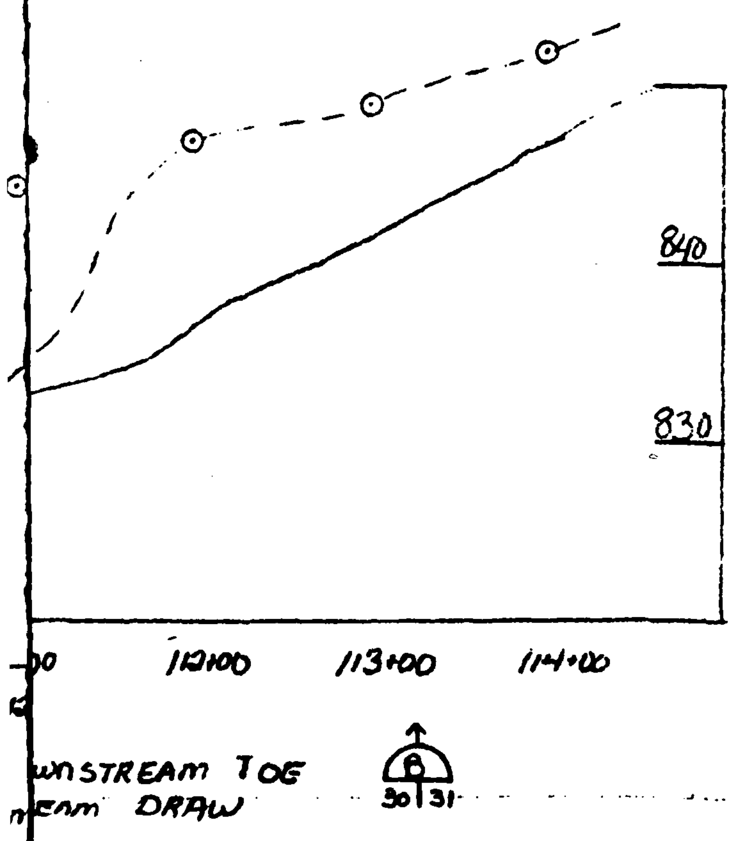
Profile OF BOTTOM
ELEVATION OF
HORIZONTAL PERVIOUS BLANKET



Profile ALONG DOWNST
AND DOWNSTREAM

15-1-7

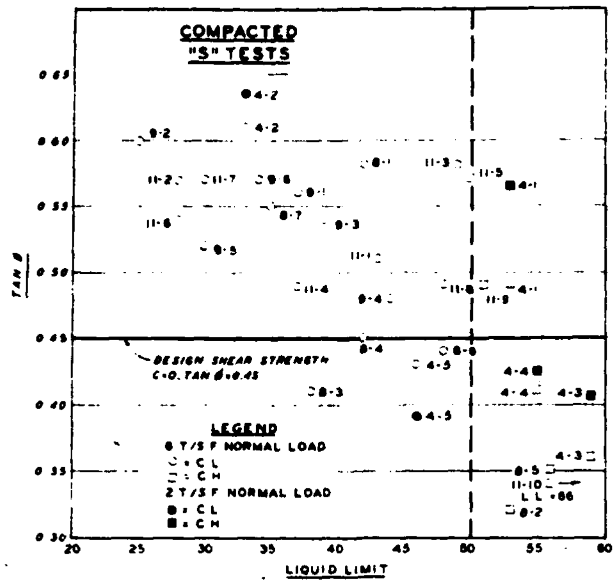
○ Piezometric Level
Projected for
Spillway Crest
w/ Test Relief
wells
(see plate 22)

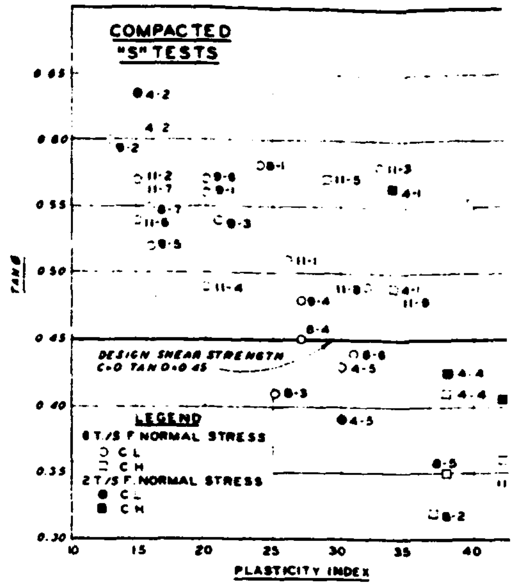


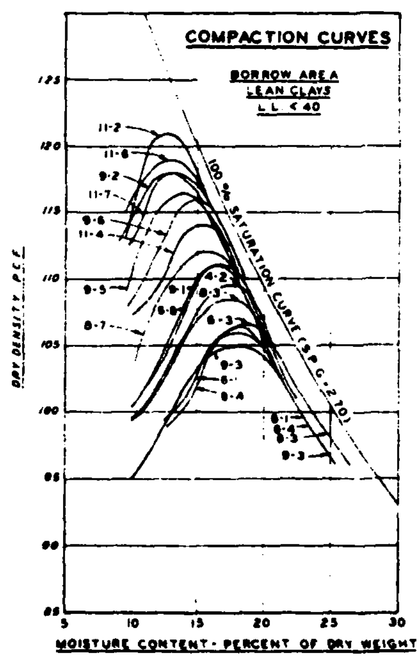
Smithville Dam
Left Abutment
Proposed Pressure Relief
Well Location
Profile

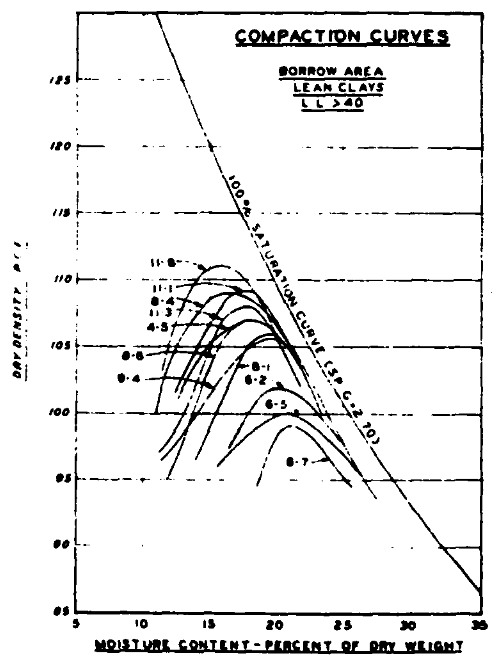
APPENDIX A
PLATE 31

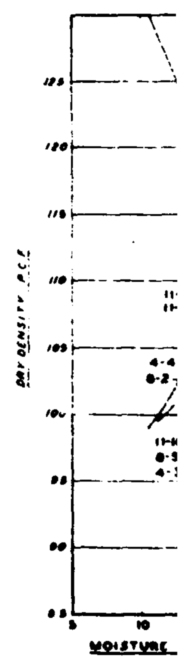
RP-3-1705
JULY 1984


COMPACTED
"S" TESTS
DESIGN SHEAR STRENGTH
C=0, TAN Ø=0.45
LEGEND
1 T/S F NORMAL LOAD
○ = C L
□ = C H
2 T/S F NORMAL LOAD
● = C L
■ = C H
TAN Ø
LIQUID LIMIT


COMPACTED
"S" TESTS
DESIGN SHEAR STRENGTH
C=0 TAN Ø=0.45
LEGEND
1 T/S F NORMAL STRESS
○ C L
□ C H
2 T/S F NORMAL STRESS
● C L
■ C H
TAN Ø
PLASTICITY INDEX

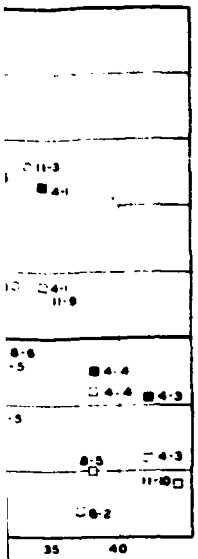

COMPACTION CURVES
BORROW AREA
LEAN CLAYS
L L < 40
100 % SATURATION CURVE (S P G = 2.70)
DRY DENSITY P.C.F.
MOISTURE CONTENT - PERCENT OF DRY WEIGHT

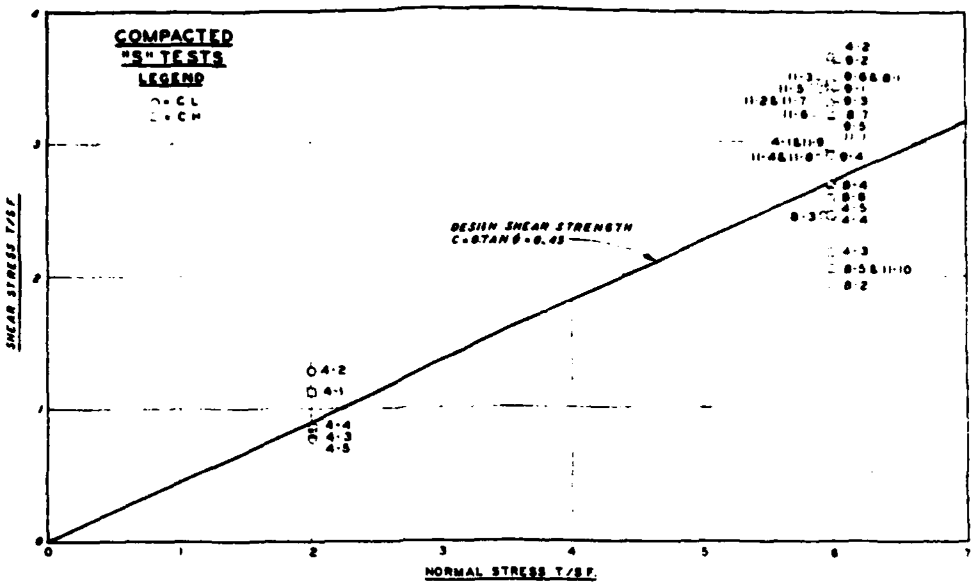

COMPACTION CURVES
BORROW AREA
LEAN CLAYS
L L > 40
100 % SATURATION CURVE (S P G = 2.70)
DRY DENSITY P.C.F.
MOISTURE CONTENT - PERCENT OF DRY WEIGHT

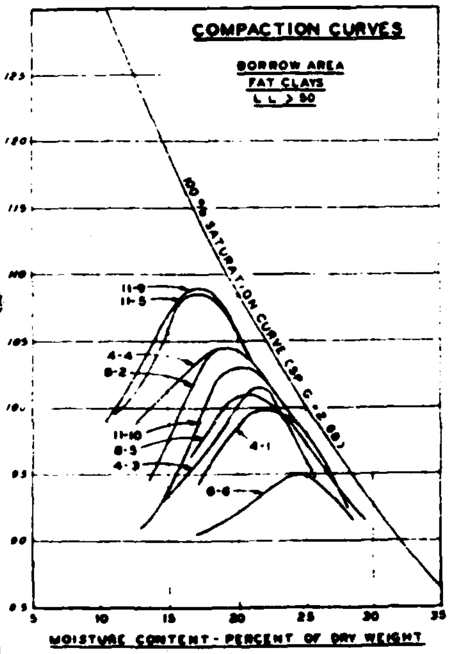


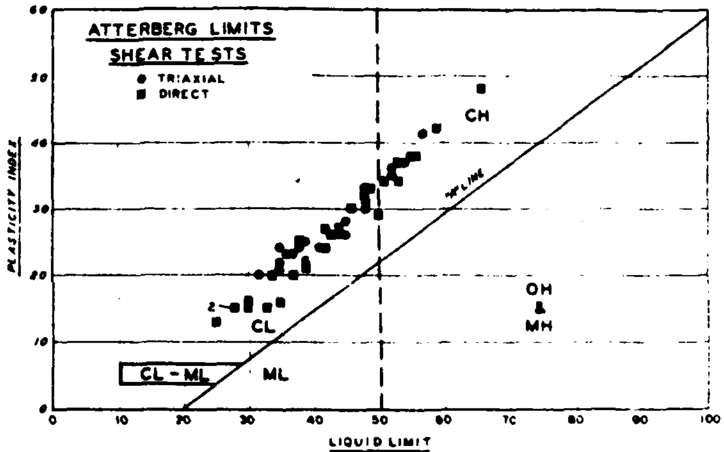
15-1-7

1









NOTE
See Plate 48 for composite identification.

SYM	DESCRIPTION	DATE	APP'D

REVISIONS

PLATTE RIVER BASIN

SMITHVILLE RESERVOIR

LITTLE PLATTE RIVER

TEST DATA SUMMARY
COMPACTED EMBANKMENT MATERIALS

In 2 sheets   Sheet No. 1   Scale: as shown

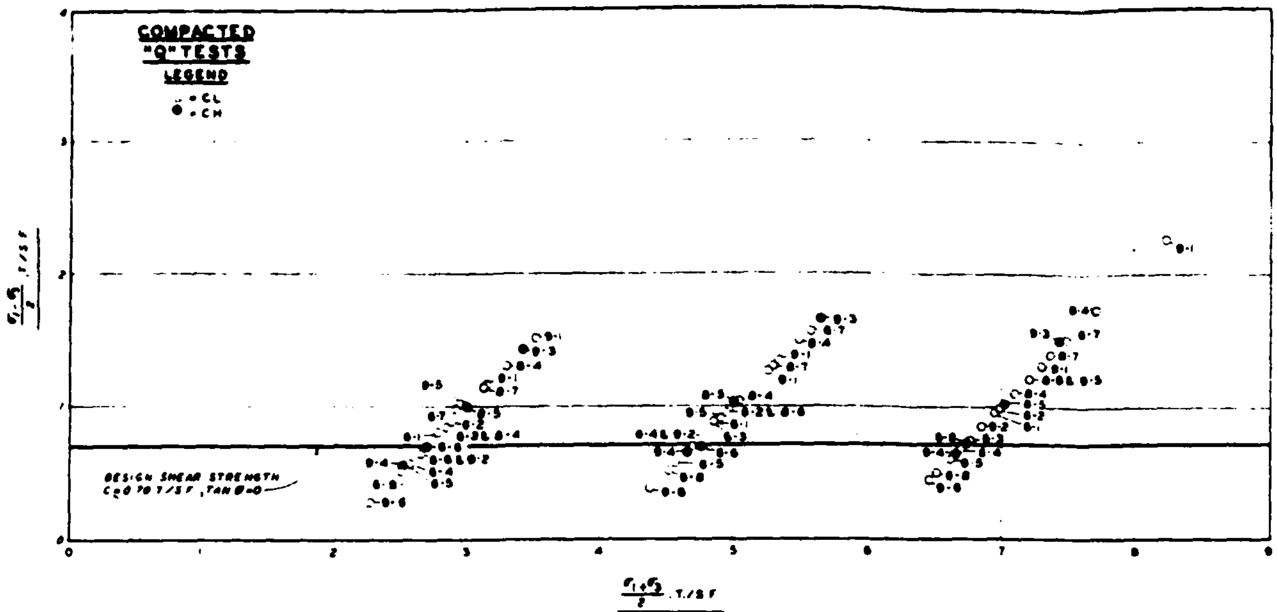
CORPS OF ENGINEERS
KANSAS CITY DISTRICT

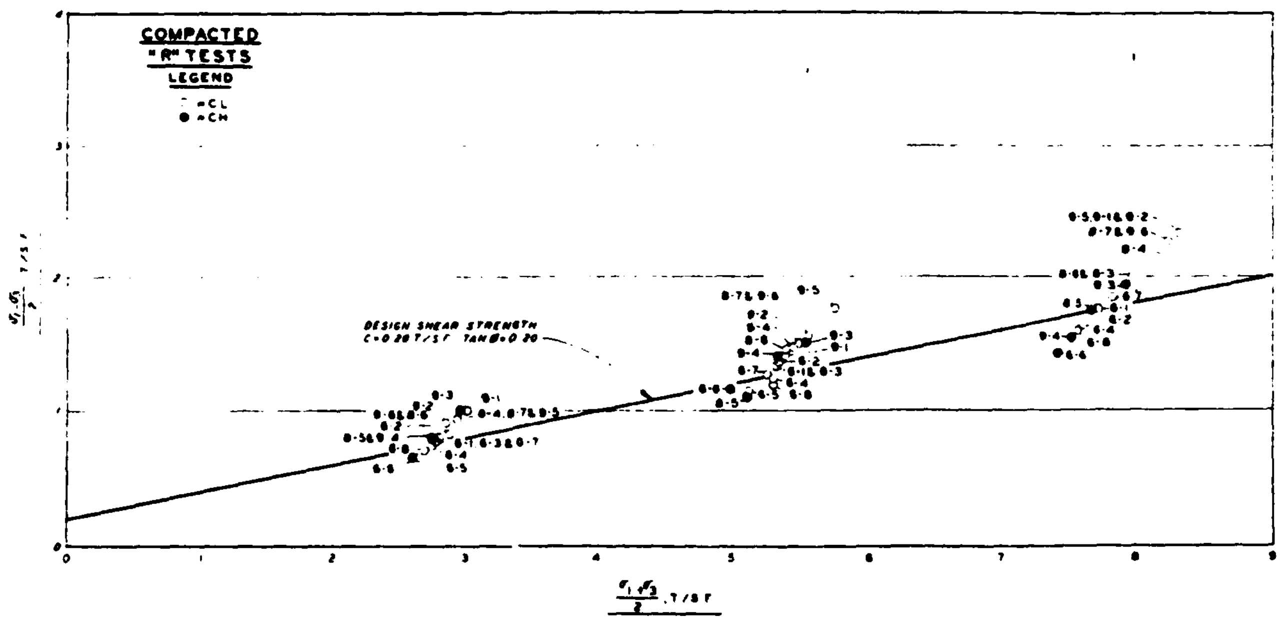
U.S. ARMY
OCTOBER 1970

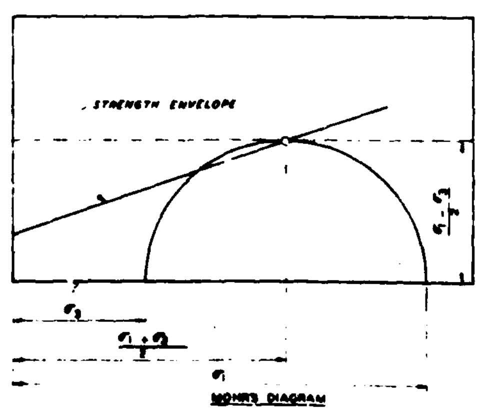
Submitted   Recommended   Approved

R.K.B.   L.E.M.   R.G.F.   DM-10

RP-7-1706   APPENDIX A   PLATE NO. 32



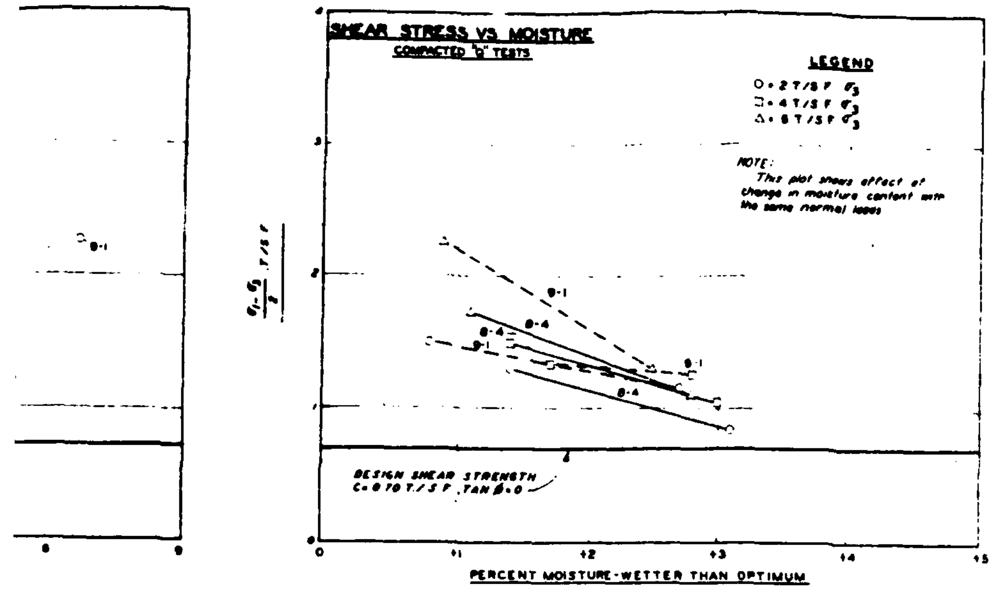


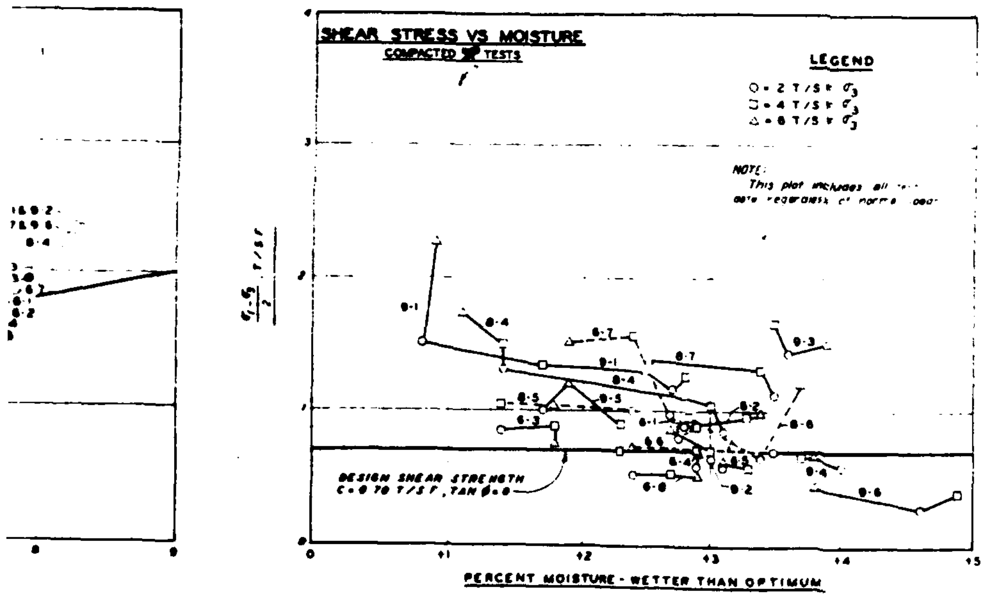


MOHRS DIAGRAM

1-7

1





NOTE
See Plate 48 for composite identification

SYM	DESCRIPTION	DATE	APP'D

REVISIONS

PLATTE RIVER BASIN

SMITHVILLE RESERVOIR

LITTLE PLATTE RIVER

TEST DATA SUMMARY
COMPACTED EMBANKMENT MATERIALS

In 2 sheets — Sheet No 2 — Scale as shown

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

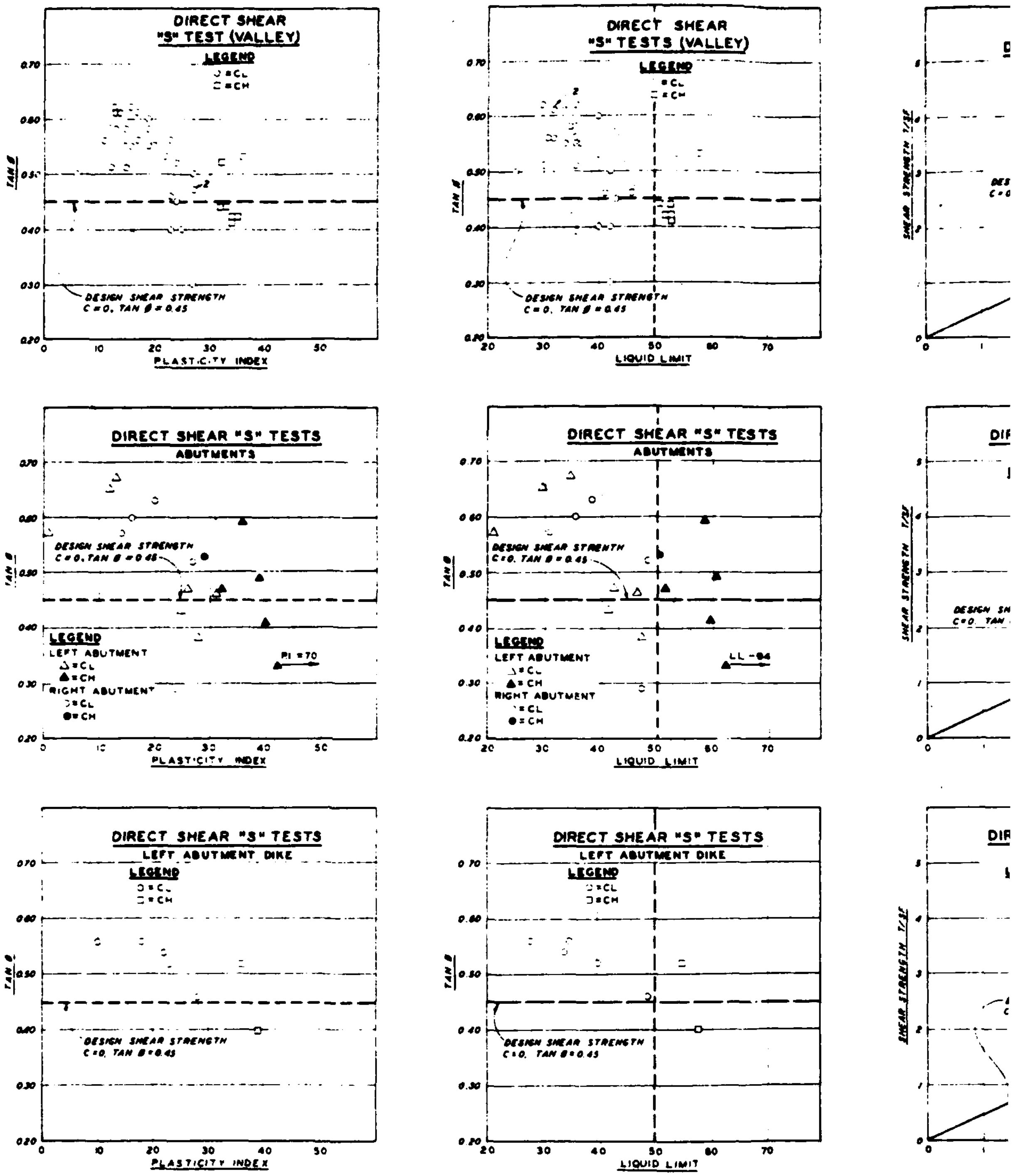
U. S. ARMY
OCTOBER 1970

Submitted / Recommended / Approved

Compiled by: R K B — Drawn by: L E M — Checked by: R G F — DM-10

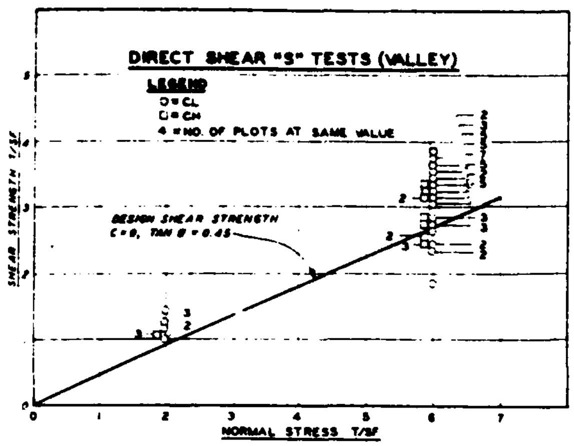
1

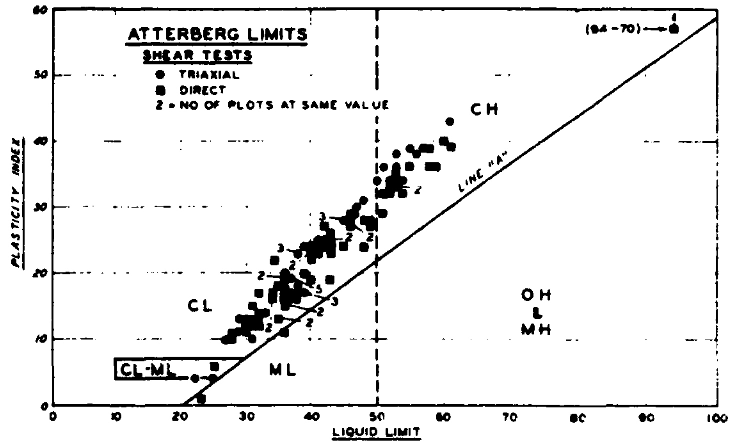
2

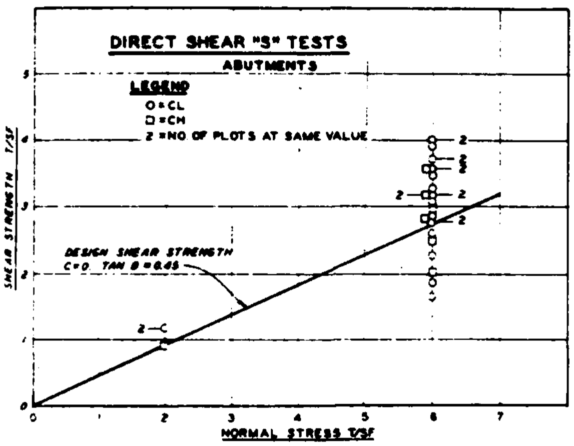


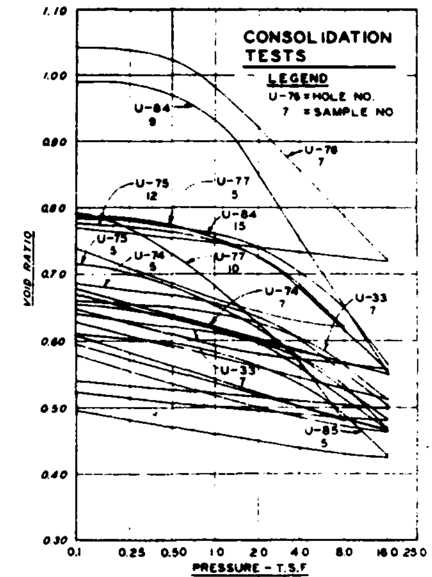
DIRECT SHEAR "S" TEST (VALLEY)
LEGEND
○ = CL
□ = CH
TAN Ø
DESIGN SHEAR STRENGTH C = 0, TAN Ø = 0.45
PLASTICITY INDEX
DIRECT SHEAR "S" TESTS (VALLEY)
LIQUID LIMIT
DIRECT SHEAR "S" TESTS ABUTMENTS
LEFT ABUTMENT
△ = CL
▲ = CH
RIGHT ABUTMENT
○ = CL
● = CH
PI = 70
LL = 94
DIRECT SHEAR "S" TESTS LEFT ABUTMENT DIKE
SHEAR STRENGTH T/SF

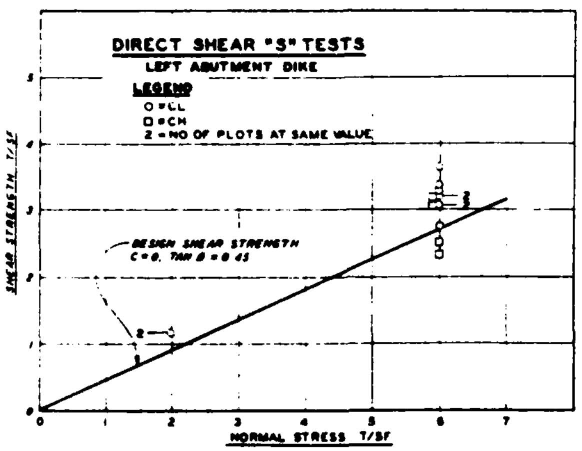
15-1-7











CONSOLIDATION TEST SUMMARY									
HOLE NO	DEPTH	LL %	Po TSF	Pc TSF	Cc	Cv (IN²/MIN) 2 TSF	4 TSF	8 TSF	16 TSF
U-33	14.8-16.8	43	1.0	3.5	.18	.0049	.0041	.0035	.0035
U-33	14.8-16.8	43	1.0	3.1	.19	—	.0098	.0041	.0035
U-74	8.0-9.6	32	.55	2.7	.20	—	—	—	—
U-74	12.2-13.9	34	.80	2.1	.19	—	—	.0164	.0131
U-75	8.0-9.9	49	.55	2.8	.37	—	—	.0031	.0017
U-75	8.0-9.9	45	.55	1.9	.23	—	—	.0035	—
U-75	22.6-23.9	38	1.1	2.3	.23	—	—	.0189	—
U-76	12.3-13.9	36	.55	0.8	.24	—	—	—	—
U-77	17.9-19.0	28	.90	—	.172	—	—	—	—
U-84	16.0-17.9	29	.90	1.0	.38	—	—	.0390	—
U-84	28.0-29.9	40	1.1	4.1	.33	—	—	—	.0350
U-85	8.3-9.0	35	.55	3.0	.23	—	—	—	.0150

SYM DESCRIPTION DATE APP'D
REVISIONS

PLATTE RIVER BASIN
SMITHVILLE RESERVOIR
LITTLE PLATTE RIVER

TEST DATA SUMMARY
FOUNDATION OVERBURDEN

In 3 sheets | Sheet No. 1 | Scale as shown

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

U.S. ARMY
OCTOBER 1970

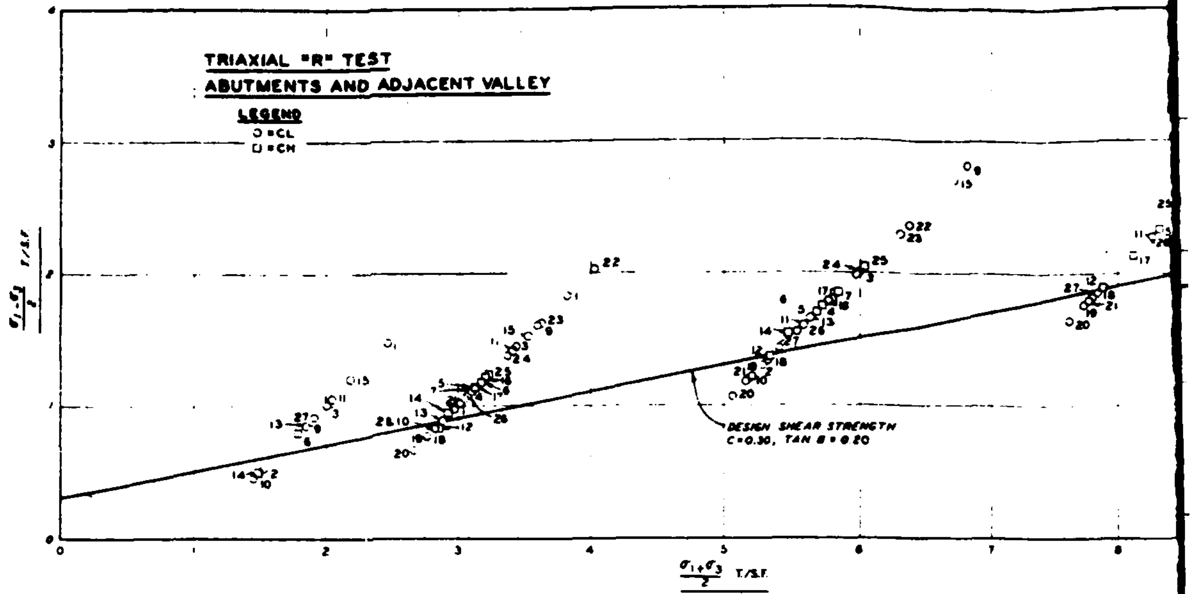
Submitted | Recommended | Approved

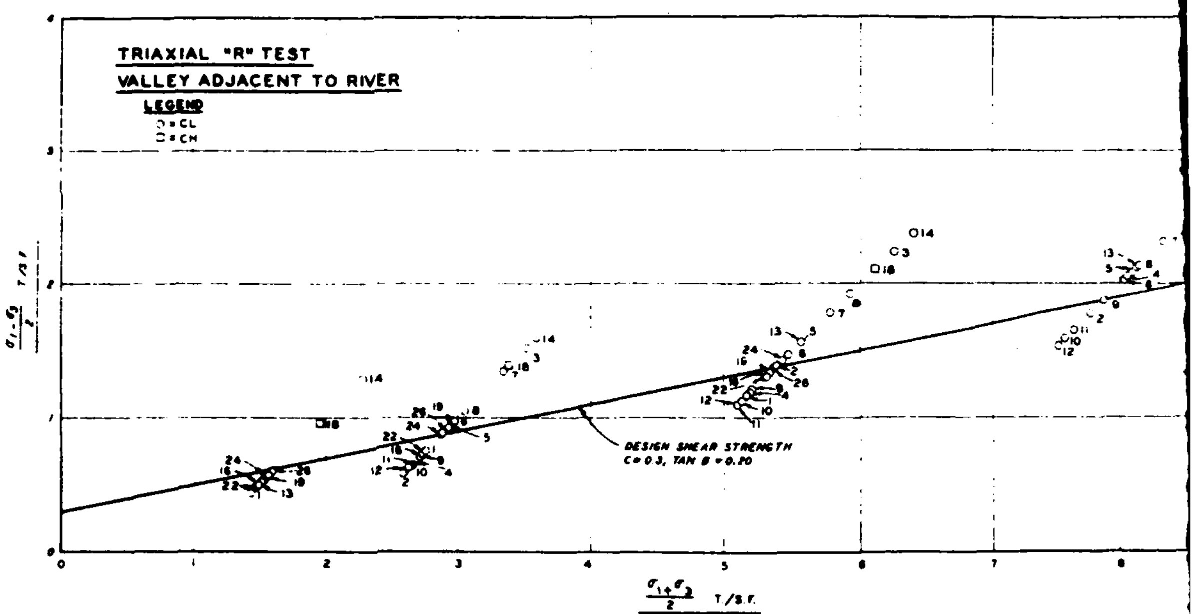
C D Gadden

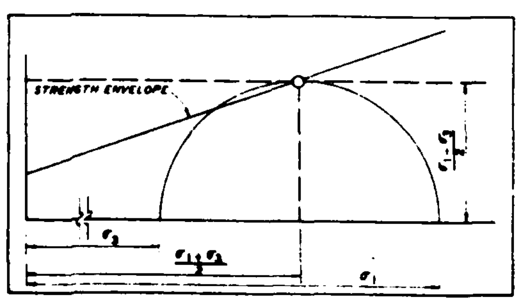
CHIEF, FOUNDATIONS & MATERIALS BR. | CHIEF, ENGINEERING DIVISION

COMPILED BY: RKB | DRAWN BY: CFL | CHECKED BY: RGF | DM 10

RP-3-1708 APPENDIX A PLATE NO. 3

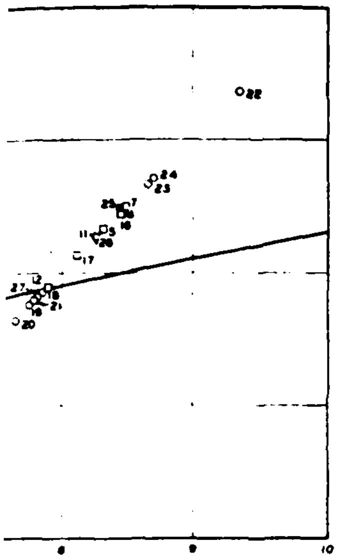






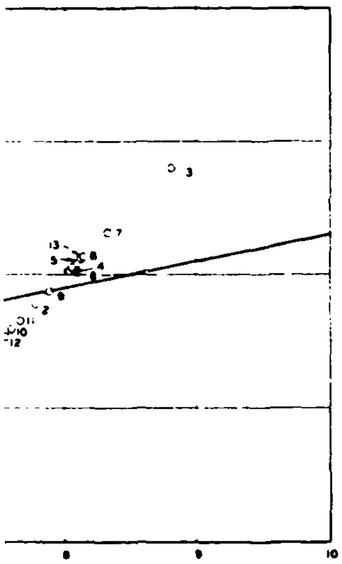
MOHR'S DIAGRAM

15-1-7



"R" TEST DATA
ABUTMENTS AND ADJACENT VALLEY

SYMBOL	HOLE NO	SAMPLE DEPTH	CLASSIFICATION SYMBOL	LL	PI	RANGE [illegible]	[illegible]
1	[illegible]	11.0-12.9	CL	38	[illegible]	[illegible]	[illegible]
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	18.4-20.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	4.2-5.9	CH	[illegible]	39	[illegible]	[illegible]
5	[illegible]	6.1-7.9	CH	[illegible]	36	[illegible]	[illegible]
6	[illegible]	4.7-5.9	CH	[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	8.2-10.[illegible]	CH	[illegible]	[illegible]	[illegible]	[illegible]
9	[illegible]	16.0-17.9	CL	[illegible]	[illegible]	[illegible]	[illegible]
10	[illegible]	20.0-21.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	U-78	2.0-3.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	U-78	2.0-3.9	CH	61	41	[illegible]	[illegible]
13	U-80	0.6-1.9	[illegible]	45	26	[illegible]	[illegible]
14	U-80	4.0-5.9	CH	[illegible]	[illegible]	[illegible]	[illegible]
15	U-80	8.0-10.[illegible]	[illegible]	48	[illegible]	[illegible]	[illegible]
16	U-84	4.1-5.9	CH	[illegible]	[illegible]	[illegible]	[illegible]
17	U-84	6.0-7.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
18	U-84	8.0-9.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
19	U-84	10.0-[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
20	U-84	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
21	U-84	22.0-23.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
22	U-85	0.5-1.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
23	U-85	[illegible]	[illegible]	38	[illegible]	[illegible]	[illegible]
24	U-85	6.0-7.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
25	U-86	0.3-1.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
26	U-86	4.2-5.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
27	U-86	6.0-7.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]



"R" TEST DATA
VALLEY ADJACENT TO RIVER

SYMBOL	HOLE NO	SAMPLE DEPTH	CLASSIFICATION SYMBOL	[illegible]	[illegible]	RANGE [illegible] CONT[illegible]	[illegible]
1	[illegible]	14.0-15.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	[illegible]	2.0-3.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	4.0-5.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	9.5-11.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	16.[illegible]-17.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	20.[illegible]-21.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	24.0-25.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	[illegible]	2.0-3.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	[illegible]	4.0-5.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	[illegible]	[illegible]-7.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	[illegible]	[illegible]-9.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	[illegible]	10.0-11.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
13	[illegible]	16.0-17.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
14	[illegible]	0.6-[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
16	[illegible]	6.0-8.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
18	[illegible]	2.0-[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
19	[illegible]	[illegible]-7.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
22	[illegible]	20.[illegible]-21.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
24	[illegible]	28.0-29.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
26	[illegible]	36.0-37.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

SYM | DESCRIPTION | DATE | APP'D

REVISIONS

PLATTE RIVER BASIN

SMITHVILLE RESERVOIR

LITTLE PLATTE RIVER

TEST DATA SUMMARY
FOUNDATION OVERBURDEN

In 3 sheets | Sheet No 3 | Scale as shown

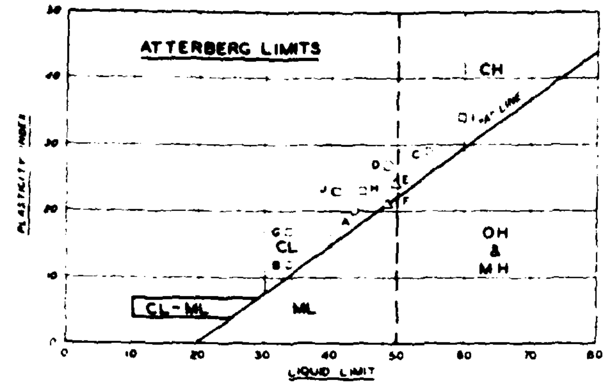
CORPS OF ENGINEERS
KANSAS CITY DISTRICT

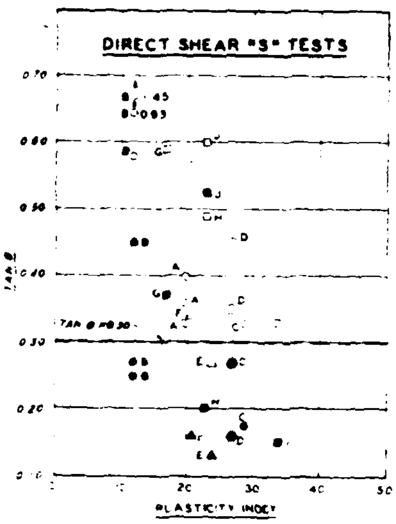
U. S. ARMY
OCTOBER 1970

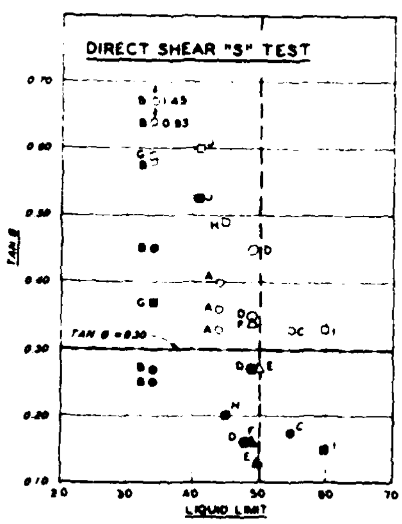
Submitted: [signature] | Recommended: E.D. Gaddis [signature] | Approved:

COMPILED BY: W W B | DRAWN BY: C F L | CHECKED BY: R K B | DM 10

RP-3-1709 APPENDIX A PLATE NO 35


ATTERBERG LIMITS
PLASTICITY INDEX
LIQUID LIMIT
CH
"A" LINE
CL
CL-ML
ML
OH
&
MH


DIRECT SHEAR "S" TESTS
TAN θ
TAN θ = 0.30
PLASTICITY INDEX


DIRECT SHEAR "S" TEST
TAN θ
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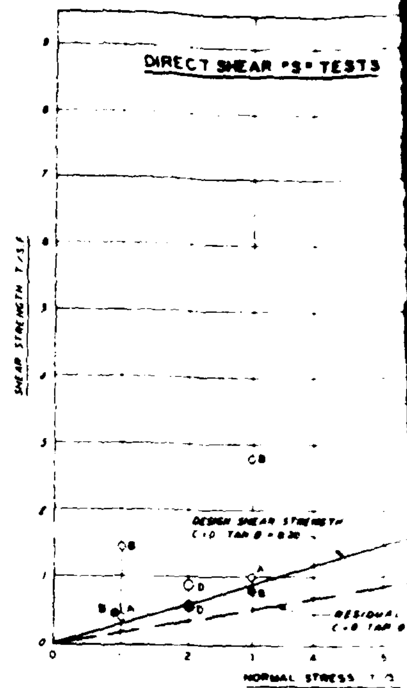
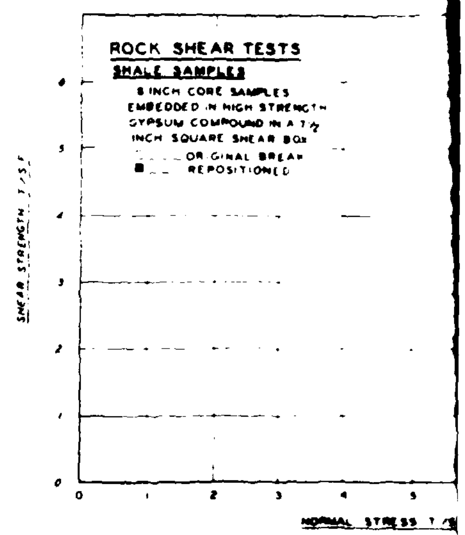
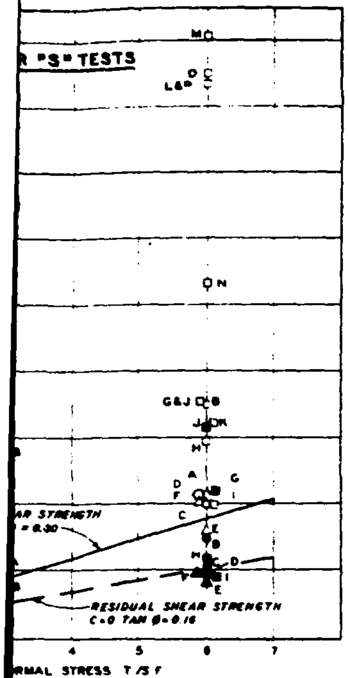
LIQUID LIMIT


DIRECT SHEAR "S" TESTS
SHEAR STRENGTH T/S.F.
DESIGN SHEAR STRENGTH
NORMAL STRESS


ROCK SHEAR TESTS
SHALE SAMPLES
6 INCH CORE SAMPLES
EMBEDDED IN HIGH STRENGTH
GYPSUM COMPOUND IN A 7½
INCH SQUARE SHEAR BOX
ORIGINAL BREAK
REPOSITIONED
SHEAR STRENGTH T/S.F.
NORMAL STRESS

15-1-7



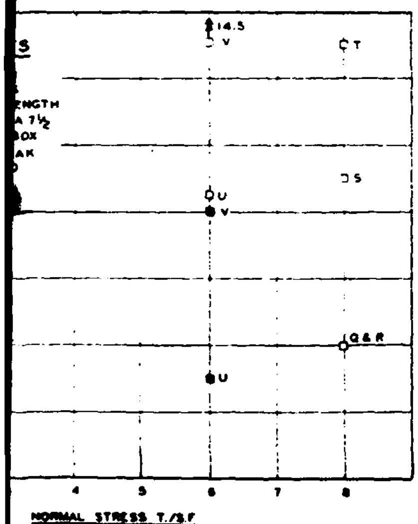
DIRECT SHEAR "S" TEST DATA

SYMBOL	SAMPLE NO.	SAMPLE DEPTH	CLASSIFICATION LETTER	LL	PI	RANGES INITIAL DRY DENSITY	INITIAL % MOISTURE	PERCENT SATURATION	GEOLOGIC MEMBER
			VALLEY						
A	UC-27-26	52.4-52.8	CL	44	20	106.0-107.5	17.5-19.5	[illegible]	[illegible]
B	UC-27-35	54.3-55.5	CL	34	12	139.5	8.5	96	[illegible]
C	U-102-23	46.0-47.4	CH	55	29	104.0	23.0	[illegible]	[illegible]
			SPILLWAY						
D	DUC-32-1	5.8-6.9	CL	49	27	102.5-106.0	19.5-2[illegible]	88-9[illegible]	BONNER SPRINGS
			RT. ABUTMENT						
E	U-86-8	14.0-15.9	CH	50	24	103.0	23.5	98	ISLAND CREEK
F	U-86-9	16.0-17.7	CL	49	21	109.0	20.0	95	ISLAND CREEK
			OUTLET WORKS						
G	DC-103-1	28.7-30.0	CL	34	17	113.0	18.5	95	CHANUTE
H	DC-103-2	30.0-31.1	CL	45	23	101.0	24.0	[illegible]	CHANUTE
I	DC-104-1	12.7-14.5	CH	60	34	104.0	23	[illegible]	[illegible]
J	D-105-1	22.0-23.7	CL	41	23	108.5	20.0	94	LANE
K	C-146-5	26.4-28.0				113.0	[illegible]		MUNCIE CREEK
L	C-146-8	30.5-32.5				132.0	[illegible]		MUNCIE CREEK
M	C-146-9	32.5-34.1				132.0	8.[illegible]		MUNCIE CREEK
N	C-146-11	35.5-36.3				126.5	12.[illegible]		CHANUTE
O	C-146-12	36.3-38.6				[illegible]	6.[illegible]		CHANUTE
P	C-146-13	38.6-40.4				[illegible]34.0	7.[illegible]		CHANUTE

ROCK SHEAR TEST DATA

OUTLET WORKS - SHALES

SYMBOL	SAMPLE NO.	SAMPLE DEPTH	DRY DENSITY	MOISTURE	HORIZONTAL DEFORMATION AT FAILURE (INCHES)	GEOLOGIC MEMBER
Q	DC-103-2	30-31	132		[illegible]	CHANUTE
R	DC-103-3	46-47	95		0.0[illegible]6	QUIVERA
S	DC-104-2	23-24	131	10.0	0.03[illegible]	MUNCIE CREEK
T	DC-104-3	32-33	139	7.[illegible]	0.04[illegible]	CHANUTE
U	C-146-14	40.4-41.2	124	11.[illegible]	0.0[illegible]	CHANUTE
V	C-146-4	25.2-26.4			0.084	[illegible]



LEGEND

MAXIMUM SHEAR STRESS	RESIDUAL SHEAR STRESS	LOCATION
○	●	VALLEY
○	●	SPILLWAY
△	▲	RIGHT ABUTMENT
□	■	OUTLET WORKS

SYM	DESCRIPTION	DATE	APP'D

REVISIONS

PLATTE RIVER BASIN

SMITHVILLE RESERVOIR

LITTLE PLATTE RIVER

TEST DATA SUMMARY FOUNDATION SHALES

In 1 sheet — Sheet No 1 — Scale: as shown

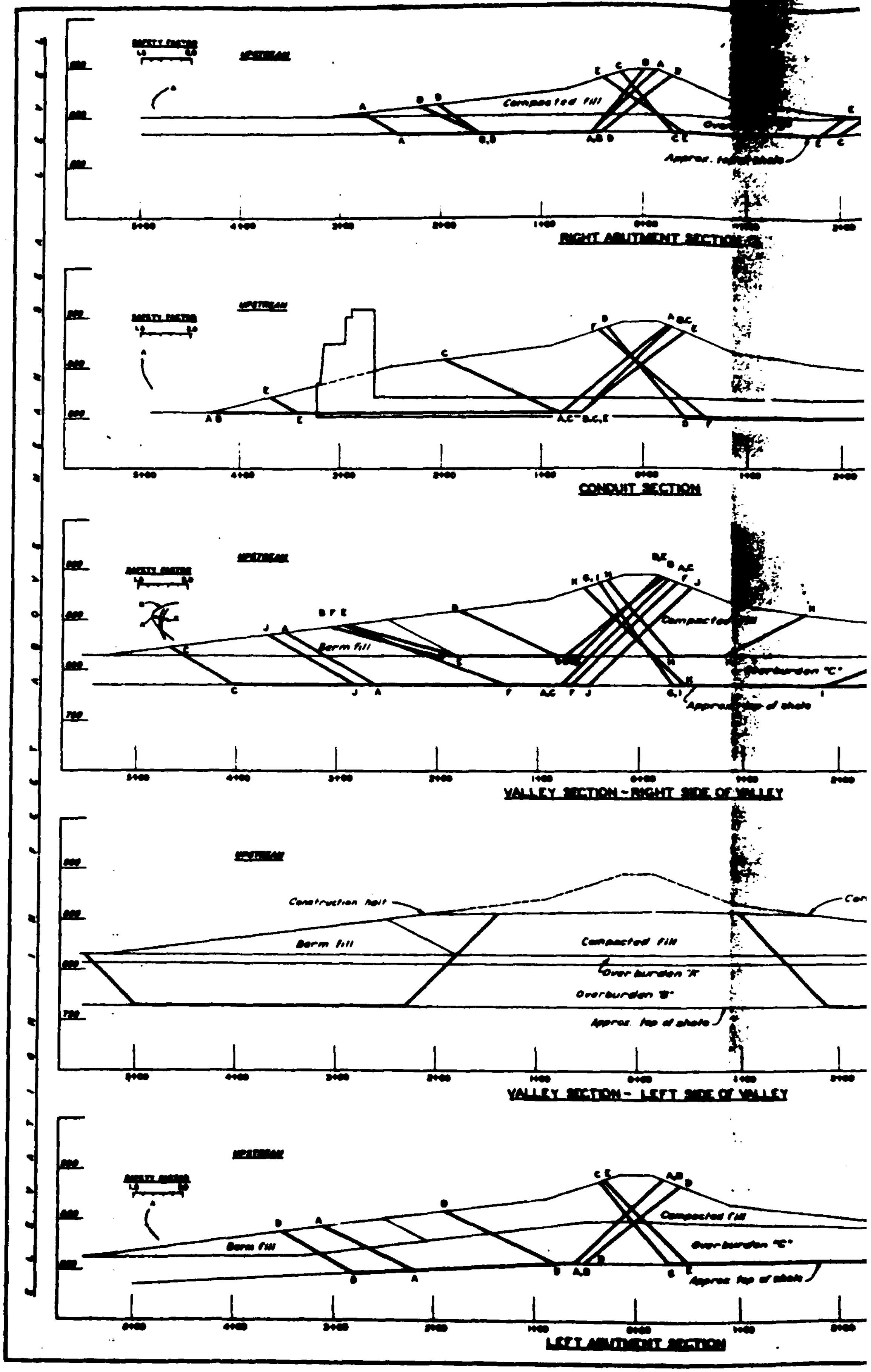
CORPS OF ENGINEERS
KANSAS CITY DISTRICT

U. S. ARMY
OCTOBER 1970

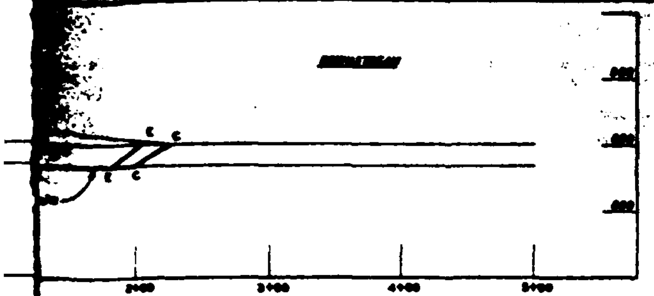
Submitted — Recommended — Approved

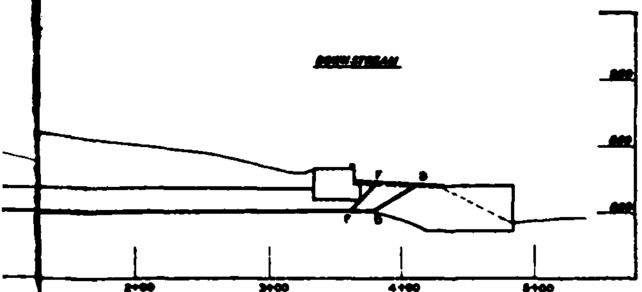
R.K.B. C.F.L. R.G.F. DM-10

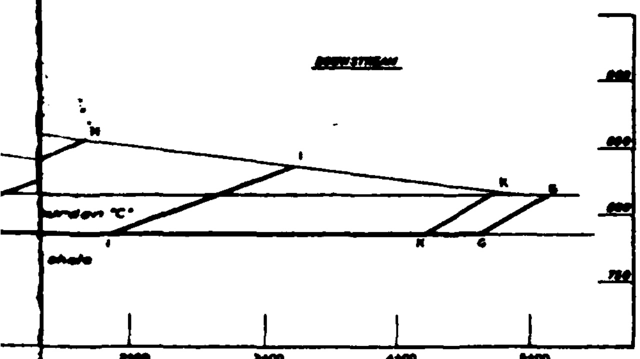
RP-3-1710 APPENDIX A PLATE NO 36

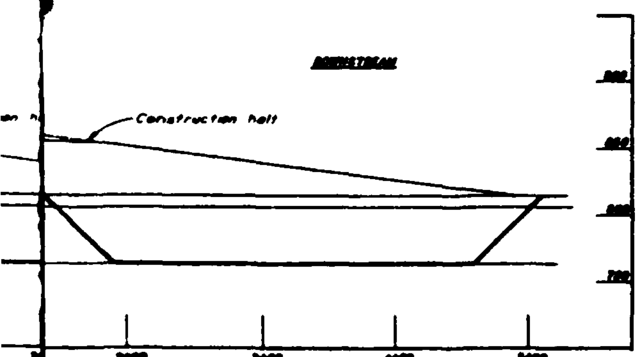

UPSTREAM
Compacted fill
Approx. top of shale
RIGHT ABUTMENT SECTION
CONDUIT SECTION
Berm fill
Overburden "C"
VALLEY SECTION - RIGHT SIDE OF VALLEY
Construction halt
Overburden "A"
Overburden "B"
VALLEY SECTION - LEFT SIDE OF VALLEY
LEFT ABUTMENT SECTION

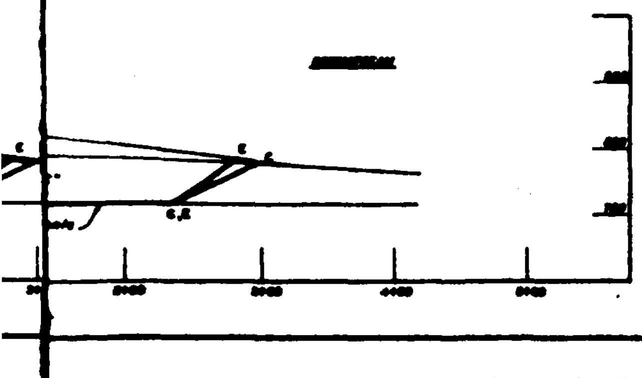
15-1-7











RIGHT ABUTMENT SECTION				
CASE	SURFACE	POOL	S.F.	
Partial Pool	A	865	1.62	(1)
Sudden Drawdown	B	[illegible]/864.2	1.31	
"	B	[illegible]/864.2	1.19	
Steady seepage	C	880.2	1.59	(1)
Construction	D		2.09	
"	E		1.87	

PHYSICAL SOIL CONSTANTS								
MATERIAL	UNIT WEIGHT LBS/FT³		DESIGN SHEAR STRENGTHS					
			[illegible]		[illegible]		[illegible]	
	[illegible]	[illegible]	C (T/FT²)	TAN Φ	C (T/FT²)	TAN Φ	C (T/FT²)	TAN Φ
Compacted fill	125	120	0.7	0.0	0.2	0.2	0.0	0.45
Berm fill	115	110	0.3	0.0	0.2	0.2	0.0	0.3
Overburden "A"	120	115	0.6	0.0	0.3	0.2	0.0	0.45
Overburden "B"	120	115	0.3	0.0	0.3	0.2	0.0	0.45
Overburden "C"	120	115	0.5	0.0	0.3	0.2	0.0	0.45
Shale contact	—	—	—	—	0.4	0.3	0.0	0.30
Shale contact (Residual)	—	—	—	—	0.4	0.3	0.0	0.16

CONDUIT SECTION			
CASE	SURFACE	POOL	S.F.
Partial Pool	A	850	1.51
Sudden Drawdown	B	[illegible]/864.2	1.30
"	B	[illegible]/864.2	1.25
"	C	"	1.25
Steady seepage	D	880.2	1.57
Construction	E		1.70
"	F		1.67

REQUIRED SAFETY FACTORS	
CASE	S.F.
Construction	1.3
Sudden Drawdown from Maximum Surcharge	1.0
Sudden Drawdown from Spillway Crest	1.2
Partial Pool	1.5*
Steady Seepage	1.5*

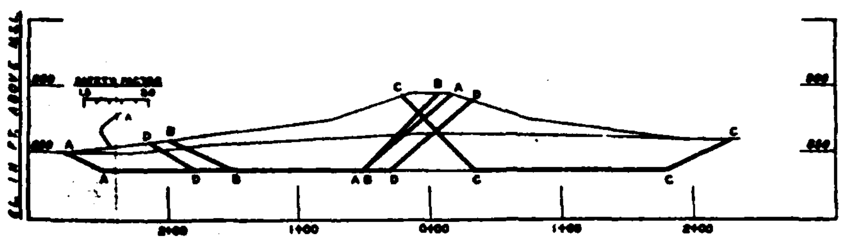
* A higher safety factor was used when the shale-overburden contact formed part of the failure surface.

Pool Elevations

Multipurpose Pool	864.2
Flood Control Pool	876.2
Spillway Crest	880.2
Maximum Surcharge	883.5

VALLEY SECTION - RIGHT SIDE OF VALLEY				
CASE	SURFACE	POOL	S.F.	
Partial Pool	A	850	1.70	(1)(2)
"	B	860	1.66	
"	C	865	1.74	(1)
Sudden Drawdown	D	[illegible]/864.2	1.24	(2)
"	E	"	1.27	
"	F	"	1.44	
"	D	[illegible]/864.2	1.15	
"	E	"	1.80	
"	F	"	1.59	
Steady seepage	G	880.2	1.62	(1)(2)
"	H	"	1.78	
"	I	"	1.84	(1)
Construction	J		1.24	
"	K		1.24	

DIKE SECTION				
CASE	SURFACE	POOL	S.F.	
Partial Pool	A	865	1.64	(1)
Sudden Drawdown	B	[illegible]/864.2	1.29	
"	B	[illegible]/864.2	1.14	
Steady seepage	C	880.2	1.59	(1)
Construction	D		1.55	



DIKE SECTION

VALLEY SECTION LEFT SIDE OF VALLEY			
CONSTRUCTION CASE			
HALT EL.	U/S SF	D/S SF	
880	1.40	1.40	
865	1.30	1.30	(2)
860	1.20	1.20	
855	0.79	0.79	

NOTES:
(1) For these cases, a higher safety factor is required because the shale-overburden contact forms part of the failure surface.
(2) These studies are the typical studies shown on Plate I.

LEFT ABUTMENT SECTION				
CASE	SURFACE	POOL	S.F.	
Partial Pool	A	860	1.63	(1)
Sudden Drawdown	B	[illegible]/864.2	1.45	
"	B	[illegible]/864.2	1.38	
Steady seepage	C	880.2	1.64	(1)
Construction	D		1.34	
"	E		1.40	

SYM.	DESCRIPTION	DATE	APPD.

REVISIONS

PLATTE RIVER BASIN

SMITHVILLE RESERVOIR

LITTLE PLATTE RIVER

EMBANKMENT STABILITY ANALYSIS SUMMARY

In 1 sheet — Sheet No. 1 — Scale: as shown

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

U. S. ARMY
OCTOBER 1970

R.F.D. — R.G.T. — R.G.F. — DM-10

RP-3-1711

APPENDIX A PLATE NO. 37

APPENDIX B

OPERATION AND MAINTENANCE MANUAL

SMITHVILLE LAKE
LITTLE PLATTE RIVER, MISSOURI

APPENDIX V

EMBANKMENT CRITERIA AND PERFORMANCE REPORT SUPPLEMENT NO. 1

APPENDIX B

SMITHVILLE DAM
LEFT ABUTMENT SEEPAGE REPORT

DECEMBER 1984

DEPARTMENT OF THE ARMY
Kansas City District, Corps of Engineers
Kansas City, Missouri

OPERATION AND MAINTENANCE MANUAL

SMITHVILLE LAKE

APPENDIX V

EMBANKMENT CRITERIA AND
PERFORMANCE REPORT

SUPPLEMENT NO. 1

LEFT ABUTMENT REMEDIAL MEASURES

APPENDIX B

CONTENTS

Tables

CONTENTS--con.

Drawings

Smithville Seepage Report

1. Introduction.--Smithville Dam is located on the Little Platte River about 1 mile northeast of Smithville, Missouri. The project was authorized by the Flood Control Act of 1965 with construction starting in February 1974. The dam is a rolled earthfill embankment with two dike sections located in low areas high on the left abutment. Foundation material consists of valley alluvium, loess residual soils and glacial drift. Impoundment began in October 1979 but lake filling was delayed because of real estate acquisition problems. Multipurpose pool, elevation 864.2, was first reached in June 1982. The record high pool to date, elevation 869.4, occurred in April 1983 and again in April 1984. Flood control pool is elevation 876.2, while spillway crest pool is elevation 880.2.

Seepage was first observed at the project during a site visit in April 1983 when the record high pool to date was reached. A seep area had developed at the downstream toe of the main embankment at Station 110+00 near the base of the left abutment. The area was spongy and wet but flowing conditions were not observed. Piezometric levels recorded in the foundation overburden in the area were 3 feet above ground. In August of the same year Mr. Roy Bowers, the owner of a tract of farmland adjacent to the left dam abutment, reported a large wet area in his pasture about 3,000 feet downstream (west) of the main dike. According to Mr. Bowers, the area had been wet during spring and early summer but he had attributed this to heavy precipitation during that period. However, during the extended dry period of the summer of 1983 the area remained wet. Consequently, Mr. Bowers reported the condition to the project office at the lake. A subsequent field reconnaissance revealed three general areas of groundwater seepage: (1) the downstream seep area; (2) the dike seep area; and (3) the Bowers seep area (as shown on Plate B-1). An investigation of the seep areas was initiated to determine the causes, potential consequences, and possible remedial measures. It included 14 piezometers and observation wells installed in a line between the lake and the Bowers seep area and in the seep area itself. Five piezometers were installed near the downstream seep area. In addition, a series of ten relief drains were installed in April 1984 on both the Bowers property and adjacent Government property to provide a measure of seepage control and to determine the effect on the piezometer surface.

2. Geology.--Smithville Lake is located near the southern limit of the Dissected Till Plains Section of the Central Lowlands Physiographic Province. Major topographic features are the maturely to submaturely developed valleys of the Little Platte River, Crows Creek, and Camp Branch. Drainage patterns typical of northern Missouri are developed on thick glacial deposits resulting in gently rolling topography. Bedrock exposures are not common but can occasionally be found along the bases of valley walls of major streams. Maximum relief in the area is about 160 feet.

a. Glacial history.--Pleistocene glaciers extended into the northern region of Missouri approximately 750,000 years ago during the Kansas glacial episode and persisted for approximately 100,000 years. Glaciers may have also advanced into the area during the earlier Nebraskan episode. Both the Nebraskan and Kansan advances were from the north-northwest and are attributed to the Iowa ice lobe from the Keewatin ice center in Canada. Since the same general regions were traversed during both episodes, the content of resultant drift materials is similar and difficult to distinguish. The southern limit of glaciation is generally recognized as being slightly south of, and approximately parallel to, the present course of the Missouri River.

Pleistocene ice sheets have been compared in size and extent to those of the Antarctic which have an average central thickness of about 6,500 feet. Estimated thicknesses of marginal masses are of the order of 1,600 feet. Glacial erosion was primarily by abrasion and quarrying whereby slabs of frozen ground were sheared from and dragged forward over nonfrozen ground. Magnitudes of erosion were dependent upon the thickness and velocity of the ice mass, the nature of materials incorporated into the basal ice, and the character of surfaces overridden. Glacial sediments include nonstratified till and, less frequently, fluvio-glacial deposits of stratified silts, sands, and gravels. Drift of variable thickness has been deposited upon essentially flat lying Pennsylvanian bedrock and is the thickest in pre-Pleistocene topographic lows.

b. Overburden.--Overburden in the vicinity of the dam is of three principal types; alluvium, glacial drift, and loess. Alluvium occupies the valleys of the Little Platte River and its tributaries and generally consists of lean and fat clays overlying clayey sands and sandy clays with minor amounts of basal gravel. Thicknesses range from 25 to 50 feet. Upland areas are deposits of glacial drift thinly mantled with loess. In the left abutment area, the drift ranges in thickness up to 85 feet and generally consists of 20 to 60 feet of till overlying 5 to 25 feet of coarser outwash sediments. Till, in general, is composed of unsorted, unconsolidated (geologically), nonstratified sediments deposited directly by and underneath glacial ice masses and consists of heterogeneous, random mixtures of clay, silt, sand, gravel, cobbles, and boulders. The overburden above approximately elevation 845 in the left abutment is predominantly lean clay glacial till with scattered gravel and cobbles and occasional isolated silty sand lenses. Below elevation 845, the material is much more heterogeneous with considerable lateral and vertical variation. Throughout most of the abutment area, the upper 11 to 20 feet of this lower unit is generally silt, however, silty clay or lean clay was encountered in some borings at this horizon. Below the silt zone, the material is coarser and consists of sand, gravel, and cobbles generally with a significant amount of silt and clay. The coarser materials underlying the till are meltwater sediments deposited from advancing or retreating ice sheets. Loess overlying the till reaches a thickness up to 20 feet in the area. The maximum thicknesses occur on broad, gently sloping, interstream divides where erosion has been minimal.

c. Bedrock.--Near surface bedrock strata are of the Pennsylvanian System, Lansing and Kansas City Groups and consist of alternating beds of shale and limestone. A geologic column for the left abutment is shown on Plate B-6. The essentially horizontal configuration of the left abutment bedrock surface is the result of a pre-Pleistocene stream channel trending generally east-west through the abutment. It is one of two major channels mapped in the reservoir area which are part of the ancestral Missouri River drainage system prior to the advance of Pleistocene glaciers. The other is located several miles upstream of the dam in the reservoir area. As ice masses traversed the area, existing sediments were scoured away and near-surface bedrock strata subjected to shear forces induced by ice thrusts.

3. Field investigations.

a. Field reconnaissance.--The initial phase of the field investigations involved a field reconnaissance of the entire left abutment region downstream of the dam embankment and main dike. Three general areas of seepage were found (see Plate B-1). The first area, labeled the Bowers seep area, covers about 20 acres of land both on Government and private property, about 3,000 feet downstream of the dike and adjacent to the valley alluvium of Wilkerson Creek. The private property includes two parcels of land, one owned by Roy Bowers and the other by Roger Burnett/Helen Cutting. The area was characterized by numerous seeps: (areas are like numbered on Plate B-2) (1) in a large draw on Government property where natural springs are located; (2) at the northeast corner of Bowers' property and extending south along the fence line; (3) above the waterline of pond A, on the Burnett/Cutting property; and (4) in the small draw to the east of pond B, the larger pond at the south boundary of the seep area. Much of the northeast corner of Bowers' pasture bounded by the draw below pond A and the draw to the west was wet and extremely soft from ponding of seepage. Flow from the area was less than 10 gallons per minute. The second area, known as the dike seep area, is located immediately downstream of Highway DD at the downstream toe of the main dike. The quantity of flowing water from this area was quite small, less than 1 gallon per minute. The third area, the downstream seep area, had previously been identified downstream of the main embankment near Station 110+00. The area had remained soft and spongy but flowing water was not evident in the area adjacent to the embankment. However, flowing water was observed in the area downstream of the toe road.

During the extended dry period in the summer of 1983, farm ponds outside the outlined seep areas were either dry or very low. Those within the seep areas were at, or very near storage capacity, being constantly fed by groundwater seepage. Also, several mature trees located in the large draw on the north side of the Bowers seep area died, apparently as a result of intolerance to the elevated zone of saturation.

In connection with the field reconnaissance, several persons familiar with the area either previous to or during construction of the dam were contacted. They related that before impoundment some of the current seep areas contained natural springs or became unusually wet during periods of wet weather. Construction personnel recalled that during the 1974 flood during which the Little Platte River valley was inundated behind the partially

completed embankment, drinking water from an old well at the construction office located on the left abutment became very dirty. It apparently became dirty as a result of the valley flooding causing a significant increase in the groundwater level.

b. Drilling and instrumentation.--A drilling program was initiated to determine if the Bowers seep area was related to the impoundment of water in Smithville Lake. The program sought information to determine the type of materials present, to determine the existing pressure gradient from the lake to the seep area, to monitor uplift pressures at the seep areas, to monitor piezometric responses to pool fluctuations over an extended period of time, and to determine the top of bedrock surface contours in the left abutment.

Initially, a series of borings were completed in the fall of 1983 along a line between the lake and the Bowers seep area (D-513 through D-518 and DC-514A) (see Plate B-1 for location and Plate B-3 for profile). Borings were advanced by continuous churn drill sampling of overburden materials with a 6-inch diameter drive barrel. Representative samples were retained for laboratory soil classification and mechanical grain size analyses. In most borings, difficult drilling conditions resulted when cobbles and gravel were encountered near the bedrock surface. In order to positively identify the bedrock surface, some of the borings were continued into bedrock short distances with NX core barrels. Borings were completed either as observation wells monitoring the more permeable basal layer of the glacial drift (D-513(OW), DC-515(OW) through D-518(OW)) or as piezometers monitoring isolated zones in the basal layers to determine if a pressure gradient existed within the basal layer itself (D-514(PZ), DC-514A(PZ)). Monitoring devices were constructed of 2-inch PVC pipe and slotted well screen (see Plate B-6 for typical installation diagrams). Table B-1 provides a listing for all the instrumentation installed during the investigations.

Five additional borings were completed downstream of the main embankment centerline to supplement and verify existing data for use in determining the extent of the more permeable basal layer (P-106-4, P-110-6, P-110-7, P-110-8, P-118-1) (see Plate B-1 and Table B-1). Drilling and sampling methods were essentially the same as the initial series. NX core samples were obtained from three holes to accurately determine top of rock. Borings were completed as piezometers with tips isolated just above top of bedrock.

A final series of eight borings were completed in February and March 1984 in the Bowers seep area to determine the thickness and material types of the alluvial confining layer and to install piezometers to measure the uplift pressures present. Three were drilled with continuous, hollow stem flight augers (A-519, A-520, A-521). Two were advanced by continuous churn drill sampling with 6-inch/4-inch diameter drive barrels (D-519A, D-521A). (These two holes were completed in the same location as the auger holes because of the need to obtain samples from the hole after the water table was encountered and to install reliable devices.) Two were drilled with a 6-inch rockbit and sampled with either a 3-inch or 1-3/8-inch split-spoon (D-522, D-523). One was drilled with a hand auger (A-524). Piezometers were installed just above bedrock in all borings but two. In A-524(PZ), the tip was isolated 8 feet below ground surface in the low permeability confining layer. A piezometer could not be successfully installed in A-519 because suspended solids left from the drilling action in the hole plugged the tip.

c. Findings.--The basal layer of the glacial drift consists of up to 40 feet of a heterogeneous mixture of silt, and silty or clayey sands and gravels, which form a natural pervious zone. This zone contains between 10 to 30 percent fines passing the No. 200 sieve (see Plate B-7). The thickness of the zone ranges from 40 feet under the upland portion of the abutment to less than 10 feet towards the valleys (see profile on Plate B-3). The zone terminates or becomes very thin in the valleys having been eroded away and replaced by alluvium. The material is more gravelly and cobbly where it is thickest, and becomes more sandy as it thins. Similar materials were found to crop out in the bluffs along Crows Creek upstream of the dam prior to lake filling. At that time it was believed the more pervious zones of glacial materials were too discontinuous and erratic to become seepage problems after impoundment. However, now it appears the more pervious zones are continuous under the entire left abutment area. The natural pervious in combination with the overlying lean clays and silts and underlying tight bedrock form a confined aquifer system which is recharged from the lake. After initial entrance pressure losses, the piezometric pressure gradient is relatively flat. The pressure gradient steepens near the seeps (see Plate B-3). The seep areas are characterized by decreasing thickness of low permeability material and pockets or lenses of more pervious material extending to near the ground surface. Pressure levels remain above ground downstream from the seeps.

4. Hydrogeologic considerations.--Prior to impoundment, the basal layer of the left abutment glacial drift was a mostly saturated unconfined aquifer system whose main discharge points were the exposures along the banks of Crows Creek along with the valley alluvium and naturally occurring springs. In 1971, three borings (D-147, D-148, and D-149) had piezometers installed in them and were monitored to investigate the possibility of abutment seepage. Two piezometers were installed in each boring, one was isolated in the basal sands and gravels and the other was isolated in higher sand lenses in the drift. The borings, however, never fully penetrated the basal layer since refusal was reached 20 feet above top of rock. Thus, when dry readings for the lower piezometers were obtained, they were misleading. As stated earlier, during the 1974 flood the construction office noticed dirty drinking water which was most likely caused by a significant rise in the water level of their water supply well. The first link was established between the influence of the river stage and the water level in the basal layer.

When impoundment began, the exposed basal material along the banks of Crows Creek became submerged. The discharge area became the recharge area initially, completing saturation of the aquifer and then subjecting the confined aquifer system to a hydrostatic pressure head corresponding to the lake level. The pressure levels increased as the lake elevation rose to multipurpose pool. Pressure levels in the basal layer near the downstream base of the abutment increased to above ground level, providing enough vertical gradient to force seepage from the basal layer through the thinning confining layer. However, since the relatively impervious alluvium blocks the aquifer as it emerges from the abutment, seepage quantities are relatively low. Flow from natural springs increased. Since the aquifer has reached a saturated condition, changes in pool level produce rapid pressures changes in the confined aquifer system. Data obtained during pool rises in the spring of 1983 indicates up to a 50 percent response in piezometric levels to changes in pool level with higher responses closer to the lake entrance point. When

pressure levels change in the confined system, the quantity or seepage will change and the extent or seep areas will tend to change. The extent or seep areas is also dependent on the length of time the pool is at a given level as well as the climatic (temperature and rainfall) conditions at the time.

5. Stability considerations.--Using the information from boring logs and piezometric data, minimum safety factors against uplift were calculated with projected piezometric levels corresponding to flood control pool and a spillway crest pool. Projections were based upon a 50 percent increase in piezometric levels corresponding to pool increases. Safety factors were computed by dividing weight of the soil mass by the uplift pressures exerted on the mass. In the Bowers seep area, the minimum safety factor against uplift was calculated as 1.1, while the area immediately downstream of the dam at Station 110+00 the minimum safety factor approached 1.0. It was concluded that the uplift stability of both seep areas was questionable when the pool reached or exceeded flood control pool.

Actions taken in the downstream area are discussed in a subsequent paragraph. In the Bowers seep area, a series of ten relief drains were installed in April 1984 to relieve and control the seepage pressures and to determine their effect on piezometer levels. Drains were initially installed on 50-foot centers and constructed with 2-inch PVC slotted well screen (.020-inch slots), (RD-1 to RD-7). The last two drains installed, RD-5A and RD-5B, were located 25 feet on either side of the RD-5 which had the largest discharge and the greatest effect on the piezometric level of the initial drains installed. These last two drains were constructed with 4-inch PVC and .030-inch slotted well screen to allow for greater discharges while reducing the entrance velocity. Each screen was surrounded by a commercial filter pack to prevent migration of the natural material into the well and to increase the effective well radius. Typical installation diagrams for both the 2-inch and 4-inch drain are shown on Plate B-6. A schedule of the relief drains are shown in Table B-2. Relief drains installed on Government property, RD-1 through RD-3, were completed with a churn rig. An all terrain vehicle mounted CME 750 was used for installation of the relief drains on Bowers' property because of extremely wet field conditions. Table B-2 includes a schedule of relief drains along with the performance of individual drains.

The effect of the relief drains on the piezometric surface was significant. Drops in pressure levels in the piezometers located down-gradient from the drains varied from in excess of 7.5 feet in D-522(PZ) to over 5.5 feet in A-521(PZ) and D-521A(PZ). Pressure drops up-gradient were measured at over 2.2 feet in D-514(PZ) and DC-514A(PZ) with the amount of pressure reductions decreasing with increasing distance from the relief drains. Table B-3 summarizes the effect of relief drains on the actual and projected piezometric levels. Since the relief drains have been installed, pool fluctuations have caused little change in the piezometric surface in the general vicinity of the wells. Piezometers located near the lake remain unaffected by the relief drains.

A temporary collector system was installed in October 1984 to contain flows from the seven relief drains on Bowers' property. It consists of 4-inch PVC pipe and connections assembled together on top of the ground. The area is fenced off to prevent livestock damage. Discharge is directed into the nearest drainage ditch to the south (below pond A). The temporary collector system has significantly dried up the area.

6. Present conditions.

a. Downstream seep area.--Seepage in the downstream area was further investigated during late spring 1984. The investigation included a stability investigation along with measures to control seepage and reduce uplift pressures. A series of four test pressure relief wells were installed in the downstream left abutment area near Station 110+00. The test wells were successful in reducing the piezometric level in the foundation. The Left Abutment Stability Report, dated July 1983, summarized the field investigations, laboratory testing, stability studies, and set forth an interim solution of three pumped wells to reduce uplift pressures during high pools. It also proposed 12 additional pressure relief wells in the downstream toe area and through the downstream embankment slope as a permanent solution to control seepage and reduce uplift pressures. The relief wells and a buried collector system were scheduled to be installed during the fall or winter 1984.

b. Dike seep area.--The draw downstream of the main dike has been soft and wet since before construction of the project. The size of willows growing in the seep areas, show that some seepage was occurring well before the lake reached multipurpose pool. However, the two seep areas are located in the two low areas where the horizontal pervious blanket of the dike extends underneath the road embankment. Instrumentation installed in the dike foundation during construction responds to pool fluctuations, but the pressures at multipurpose pool are not high enough to cause seepage to exit at the ground surface at the toe. (See Plates B-5, B-27, and B-28.) Projected pressure levels at higher pools are above ground, but the thickness of the confining layer should prevent excessive seepage and uplift pressure problems.

c. Bowers seep area.--The relief drains have significantly reduced pressure levels and maintained the lowered levels during pool rises, but piezometric levels are still slightly above the ground surface (1.5 to 2.0 feet). Some surface seepage is still occurring. The groundwater table is still at the ground surface in several places. The amount of seepage is still great enough to cause flowing conditions in the following areas: (1) in the large draw on Government property where the influence of the lake has increased the flow of natural springs; (2) on Government property near the northeast corner of Bowers property; (3) above the waterline of pond A; and (4) in the small draw to the east of pond B (areas marked on Plate B-2). All of the flow crosses Bowers' property before entering Wilkerson Creek. The hillside remains damp in several places (labeled A, B, C, and D on Plate B-2). Erosion and headcutting in the draws leading to Wilkerson Creek are being aggravated by the constant flow from the relief drains, seeps, and natural springs, (labeled I, II, III on Plate B-2). Short term leases are being acquired on both properties.

7. Conclusions.

a. Downstream seep area.--The downstream seep area is directly affected and fed by the lake. Due to the potential embankment stability problems caused by this seepage it was the subject to the Left Abutment Stability Report, dated July 1984 which provided an analysis of the problem and proposed permanent remedial action. The remedial action which includes 12

pressure relief wells and a buried collector system is presently under construction. When completed, it will provide a comprehensive and permanent solution to the seepage conditions in this area.

b. Dike seep area.--There is no apparent direct connection between lake levels and the seepage in the dike area. The present seepage is believed to be originating from isolated sand lenses in the glacial drift in the dike foundation. This seep area was present prior to construction and has not changed significantly with the filling of the lake. However, because of piezometric response to the lake in the lower overburden, this area will continue to be monitored and possibly have some additional instrumentation installed. No remedial work is warranted.

c. Bowers seep area.--The existing seepage conditions on the Bowers property, the Burnett/Cutting property, and the adjoining Government lands are caused by and affected by the lake levels. The relief drains previously installed help to reduce excessive seepage pressures. However, the associated problems with large saturated areas and headcutting still exist. Although areas of instability during high lake levels would probably not have the potentially catastrophic consequences that similar areas immediately downstream of the dam would have, they could be significant, particularly on private property. Access and remedial actions in an emergency could be severely restricted. It is unfortunate the affected Bowers and Burnett/Cutting properties were not acquired during the original land acquisition for the Smithville Lake. Due to the larger and more complex seepage conditions in this area, an engineered total remedial solution is not cost effective since any boils and/or slides caused by high lake levels would not likely be catastrophic. Remedial work consisting of a buried collector system for the existing relief drains, an emergency access trail, and rock ditch construction to halt head cutting will be an adequate engineered solution to the seepage problems, provided the affected tracts of land are acquired by the Government. Government acquisition will insure the necessary access for monitoring, instrumentation, additional remedial work, and emergencies. The cost estimate for the minimal remedial work necessary consisting of the buried collector system, an emergency access trail, and the rock ditch protection is $70,000.

8. Recommendation.--The 20 acres of Bowers property and 20 acres of the Burnett/Cutting property should be acquired by the Government. The minimal remedial work consisting of the buried collector system, an emergency access trail and rock ditch protection should be constructed. This area should be closely monitored, especially during high pools, and additional remedial work be performed as needed.

TABLE 1 Instrumentation Schedule

Number	Station	Range	Tip Elev.	Grd. Elev.	Tip Matl.	Installation Date	Type riser / tip	Remarks
EMBANKMENT								
P-106-4	106+02	0+15 DS	807.9	894.9	Sly Grv Sd*	12-09-83	3/4" / 1.5' - .010"	
P-110-6	110+00	3+48 DS	808.1	838.1	Sly Sdy Grv	10-26-83	2" / 1' - .020"	Flowing hole
P-110-7	110+00	4+23 DS	809.5	832.5	Cly Grv Sd*	11-22-83	2" / 1' - .020"	Flowing hole
P-110-8	110+00	1+09 DS	811.0	863.0	Cly Sdy Grv*	11-17-83	3/4" / 1.5' - .010"	
P-118-1	118+00	1+00 DS	807.0	868.5	Sly Grv Sd	11-08-83	3/4" / 1.5' - .010"	
ABUTMENT								
D-513(OW)			810.0	851.8	Sly Grv Sd/ Sly Sdy Grv	09-21-83	2" / 10' - .020"	
D-514(PZ)			817.7	841.0	Cly Grv Sd/ Sly Sd	10-13-83	2" / 2' - .020"	Flowing hole
DC-514A(PZ)			809.9	841.4	Cly Grv Sd*	10-14-83	2" / 2' - .020"	Flowing hole
D-515(OW)			807.6	877.1	Sly Grv Sd/Sly Sd Grv/Cly Grv Sd	10-28-83	2" / 10' - .020"	
D-516(OW)			812.7	892.9	Sly Grv Sd/Sly Sdy Grv/Cly Grv Sd	10-03-83	2" / 10' - .020"	
DC-517(OW)			812.9	880.0	Sly Grv Sd	10-07-83	2" / 5' - .020"	
D-518 (OW)			823.3	896.3	Sly Grv Sd/Cly Grv Sd	10-20-83	2" / 5' - .020"	

TABLE 1 (Cont'd)

Number	Station	Range	Tip Elev.	Grd Elev.	Tip Matl.	Installation Date	Type riser / tip	Remarks
BOWERS SEEP AREA								
A-519			-	839.5	-	-	-	PZ not installed, suspended solids in hole
D-519A(PZ)			820.9	838.9	Sdy Cl*	04-02-84	3/4" / 1.6' - .020"	
A-520(PZ)			812.1	828.6	Sdy Cl*	02-09-84	3/4" / 1.5' - .010"	
A-521(PZ)			810.2	830.2	Sly Sd*	02-10-84	3/4" / 1.5' - .010"	Flowing hole
D-521A(PZ)			808.6	830.0	Sly Sd*	04-03-84	3/4" / 1.6' - .020"	Flowing hole
D-522(PZ)			812.2	834.1	Cly Sd*/Sd*	03-28-84	2" / 2' - .020"	Flowing hole
D-523(PZ)			793.4	821.6	Sdy Cl	03-27-84	2" / 2' - .020"	
A-524(PZ)			819.0	827.0	Lean Cl	03-29-84	2" / 2' - .020"	

* Field Classification
OW - Observation Well
P, PZ - Piezometer

TABLE 2 - Relief Drains; Schedule and Performance

Number	Grd. El.	Installation Date	Order of Installation	Type Riser/Tip	Feet of Sand Around Tip	Initial Discharge, Est.	Imed. Affect Adj. on PZ's	Discharge; After Drains Completed 10 April 84	Discharge Pool El. 863.4 4 Oct 84
RD-1	830.84	4-9-84	8	2"/4'-.020"	9.6'	1/2-1 gpm	-	.1 gpm	.1 gpm
RD-2	831.24	4-8-84	6	2"/2'-.020"	8.3'	5 gpm	>-3'(D-521A,A-521)	1.0 gpm	1.2 gpm
RD-3	832.62	4-7-84	4	2"/2'-.020"	7.0'	1-2 gpm	-.3', -.5'(D-521A,A-521)	.1 gpm	.1 gpm
RD-4	835.39	4-7-84	1	2"/2'-.020"	5.2'	4 gpm	-.2' (D-522)	.5 gpm	.3 gpm
RD-5	834.92	4-7-84	2	2"/5'-.020"	9.8'	25 gpm	-4.2' (D-522)	7.1 gpm	1.0 gpm
RD-5A	835.60	4-10-84	10	4"/5'-.030"	7.5'	4 gpm	-.2' (D-522)	6.2 gpm	3.75 gpm
RD-5B	834.33	4-9-84	9	4"/6.5'-.030"	9.2'	18 gpm	-1.2' (D-522)	7.9 gpm	6.6 gpm
RD-6	833.28	4-7-84	3	2"/5'-.020"	8.2'	20 gpm	-1.4' (D-522)	10 gpm	6.6 gpm
RD-7	832.25	4-8-84	5	2"/3'-.020"	6.6'	2 gpm	-	1.5 gpm	1.0 gpm
RD-8	830.82	4-8-84	7	2"/2.5'-.020"	7.4'	-	-	.1 gpm	.1 gpm

Relief drains were installed between 7 April 1984 and 10 April 1984. Pool level during this time rose from El. 867.7 to El. 868.4.

Table 3 Effect of Relief Drains on Piezometric Levels

Device	Piezometric Level Before Relief Drains Installed (Friday 6 April 1984)	Piezometric Level During Installation (9 Drains Installed) (Monday 9 April 1984)	Piezometric Level After Relief Drains Installed (Tuesday 10 April 1984)	Projected Piezometric Levels Tuesday 10 April 1984 (No drains)	Comparison of Pz Levels Friday Readings w/ Tuesday Actual Readings	Comparison of Pz Levels Friday Readings w/ Tuesday Projected Readings
D-513(OW)	847.8	846.85	845.65	847.9	-2.15	-2.2
D-514(PZ)	845.6	843.55	843.20	845.8	-2.4	-2.6
DC-514A(PZ)	845.9	843.95	843.65	846.1	-2.25	-2.4
D-515(OW)	850.65	850.45	850.35	850.8	-.3	-.4
D-516(OW)	853.7	853.85	853.75	853.9	+.05	-.2
D-518 (OW)	857.3	No Reading	857.7	857.6	+.4	+.1
D-519A(PZ)	838.87	838.92	838.82	838.9	-.05	-.1
A-520(PZ)	822.4	822.9	822.5	* 822.6	+.35	+.2
A-521(PZ)	838.3	832.5	832.3	838.6	-5.98	-6.2
D-521A(PZ)	837.64	832.35	832.15	**	-5.49	—
D-522(PZ)	843.78	836.62	835.93	843.9	-7.85	-7.9
D-523(PZ)	818.31	818.96	818.86	—	+.55	—
A-524(PZ)	826.76	826.91 ±	826.86	826.8	+.1	+.06

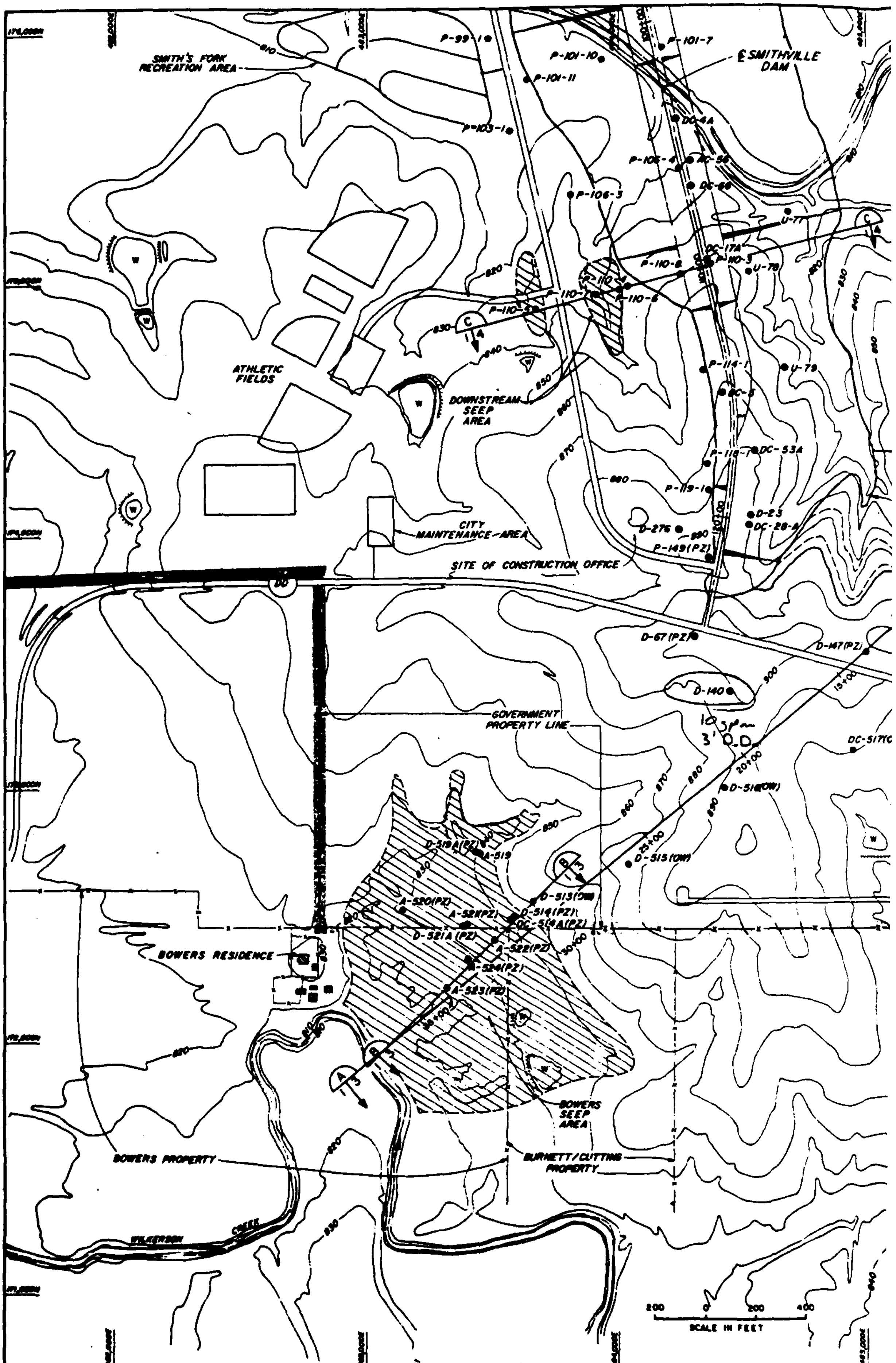
* Used 25% response
** Could not project since 6 April 1984 is only reading available before beginning drains

Pool Elevation:	6 April 1984	867.7
	9 April 1984	868.3
	10 April 1984	868.4

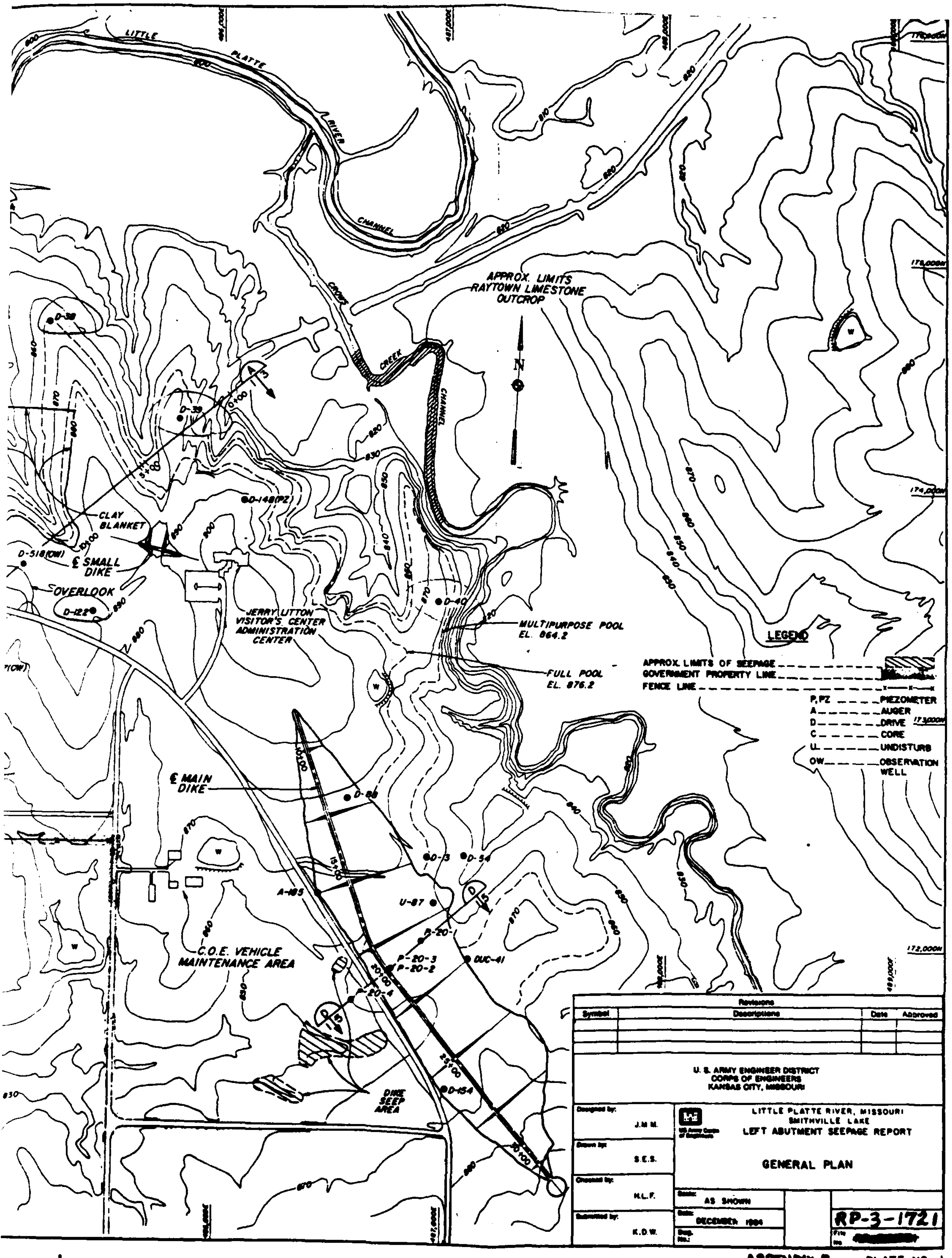
TABLE 3

DRAWINGS

DRAWINGS


SMITH'S FORK RECREATION AREA
SMITHVILLE DAM
ATHLETIC FIELDS
DOWNSTREAM SEEP AREA
CITY MAINTENANCE AREA
SITE OF CONSTRUCTION OFFICE
GOVERNMENT PROPERTY LINE
BOWERS RESIDENCE
BOWERS SEEP AREA
BOWERS PROPERTY
BURNETT/CUTTING PROPERTY
WILKERSON CREEK
P-99-1
P-101-7
P-101-10
P-101-11
P-103-1
P-106-3
P-110-6
P-110-5
P-114-1
P-119-1
P-149(PZ)
D-276
D-23
DC-53A
U-77
U-78
U-79
D-67(PZ)
D-147(PZ)
D-140
D-516(OW)
D-515(OW)
D-513(OW)
D-514(PZ)
D-519A(PZ)
A-519
A-520(PZ)
A-521(PZ)
D-521A(PZ)
A-522(PZ)
A-524(PZ)
A-523(PZ)
10 gpm
3' O.D.
SCALE IN FEET

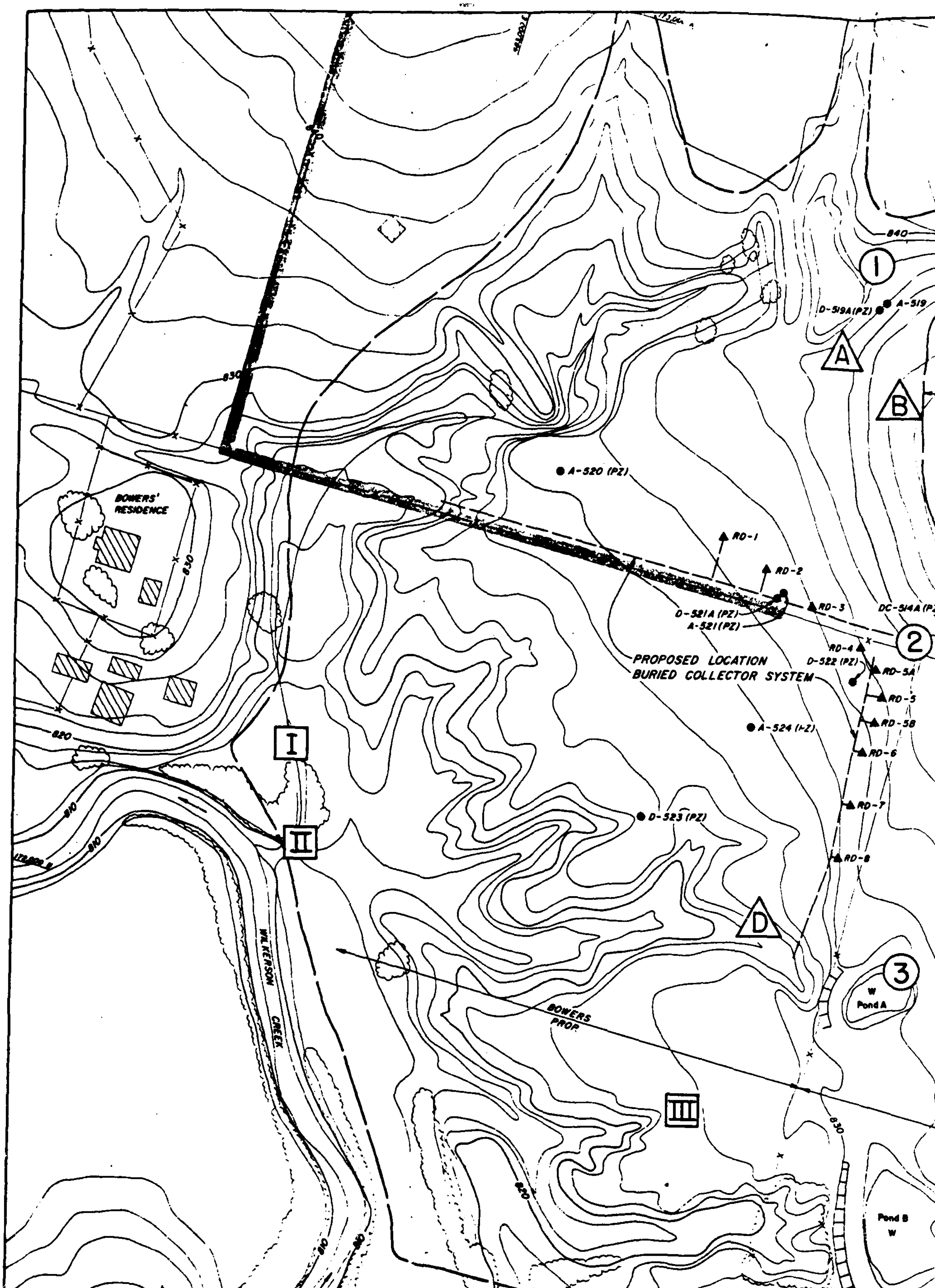
15-1-7



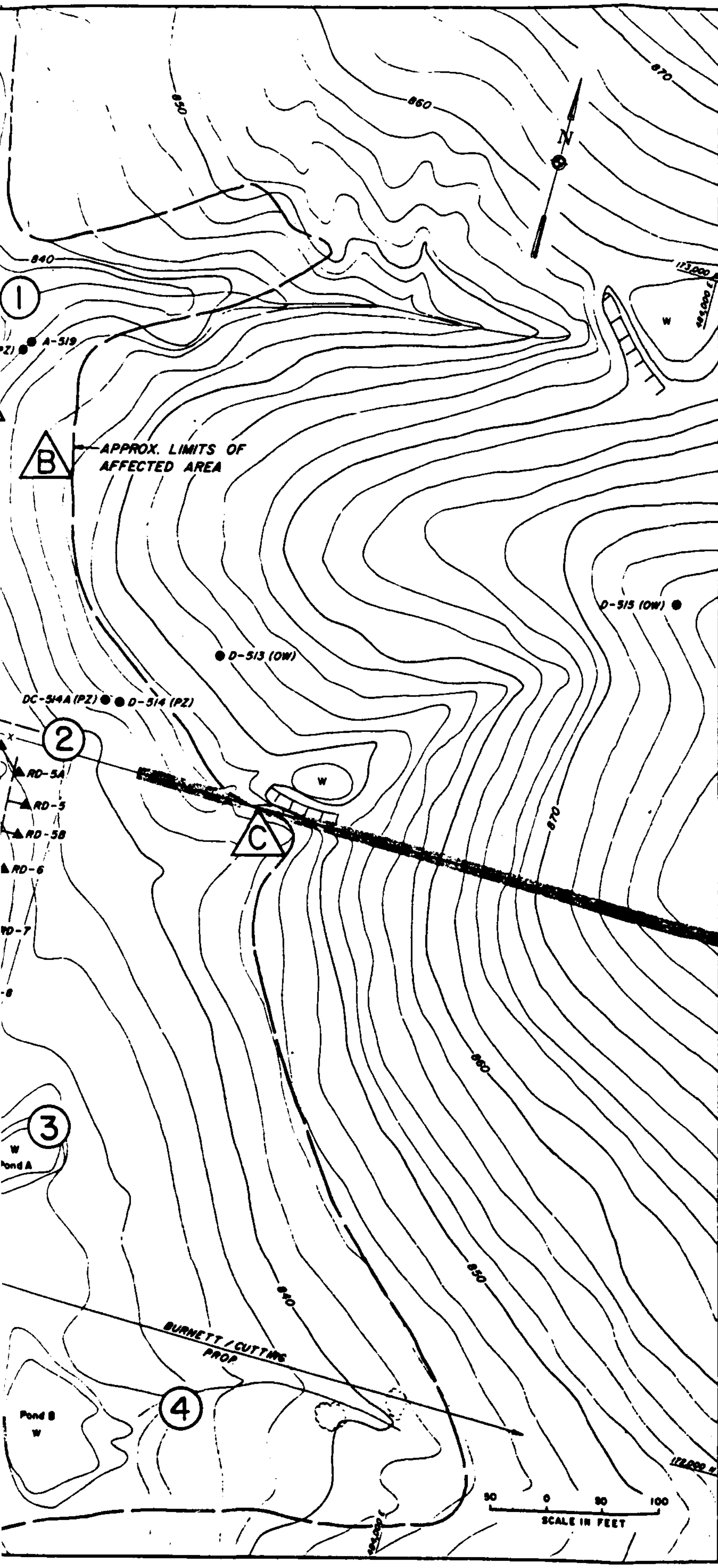
1

APPENDIX B PLATE NO. 1

2


BOWERS' RESIDENCE
830
840
820
810
A-520 (PZ)
RD-1
RD-2
RD-3
D-521A (PZ)
A-521 (PZ)
PROPOSED LOCATION
BURIED COLLECTOR SYSTEM
D-519A (PZ)
A-519
DC-514A (PZ)
RD-4
D-522 (PZ)
RD-5A
RD-5
RD-5B
RD-6
A-524 (PZ)
RD-7
RD-8
D-523 (PZ)
1
2
3
A
B
D
I
II
III
BOWERS PROP
WILKERSON CREEK
W
Pond A
Pond B
W

15-1-11



LEGEND

RD	RELIEF DRAIN
(OW)	OBSERVATION WELL
(PZ)	PIEZOMETER
▬	GOVT. PROP. LINE
①	SEEPAGE - FLOWING
▲	SEEPAGE - DAMP
ⓘ	EROSION - SEVERE HEADCUTTING

Revisions

Symbol	Descriptions	Date	Approved

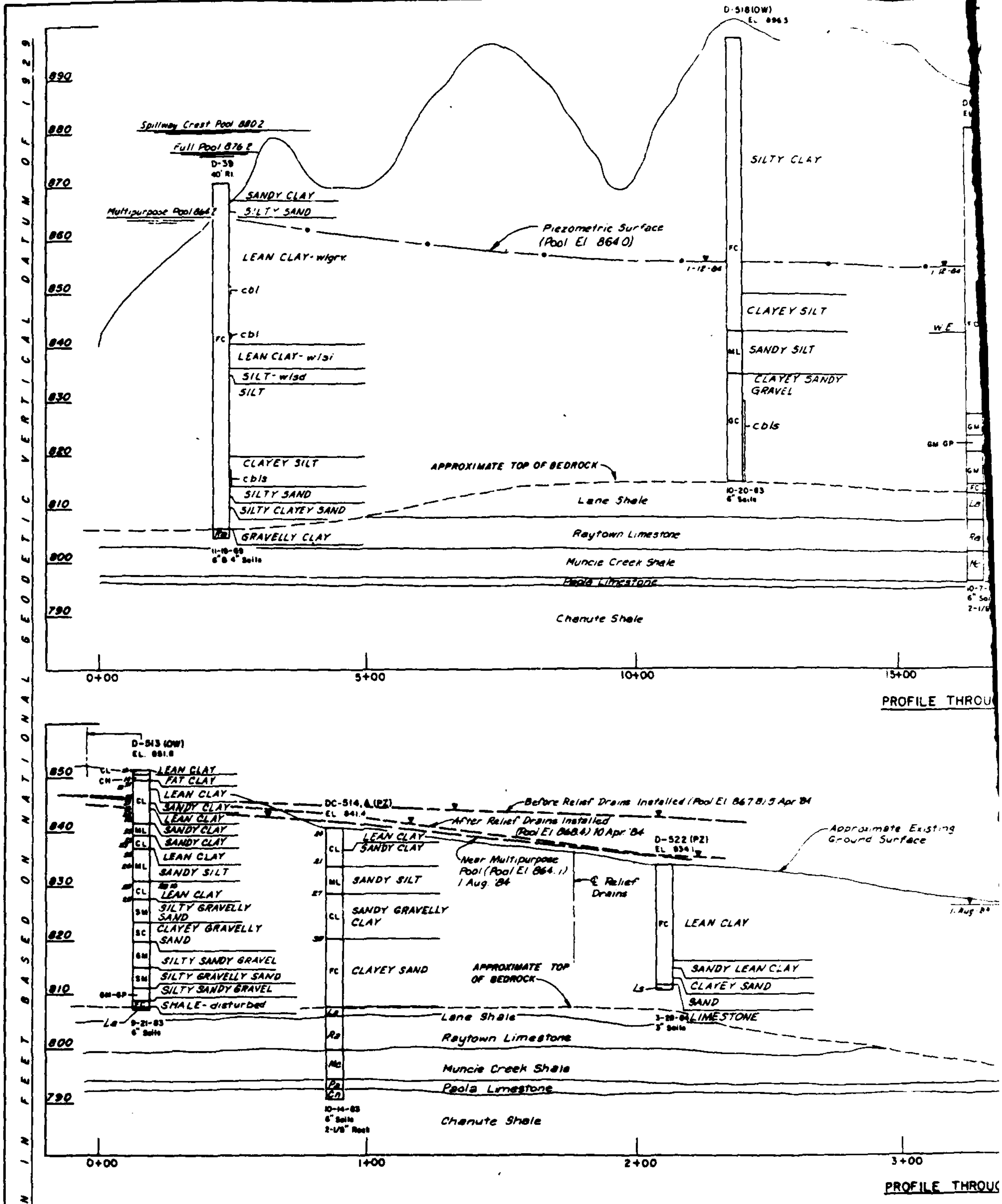
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J.M.M	US Army Corps of Engineers — LITTLE PLATTE RIVER, MISSOURI; SMITHVILLE LAKE; LEFT ABUTMENT SEEPAGE REPORT
Drawn by: J.W.P.	BOWERS SEEP AREA
Checked by: H.L.F	Scale: AS SHOWN
Submitted by: K.D.W.	Date: DECEMBER 1984 — RP-3-1722
	Dwg. No.: — File No. [redacted]

APPENDIX B PLATE NO. 2

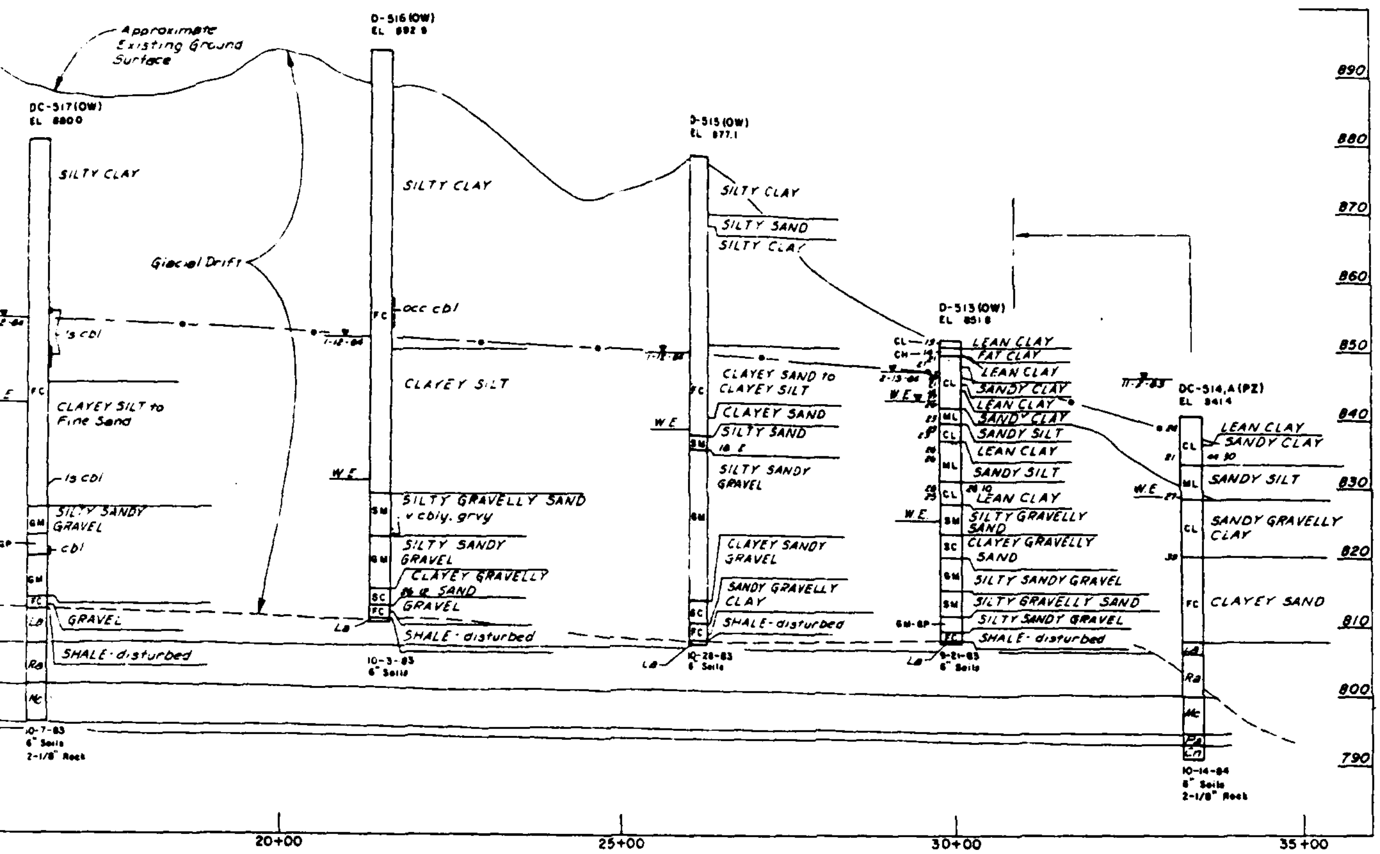
1

2

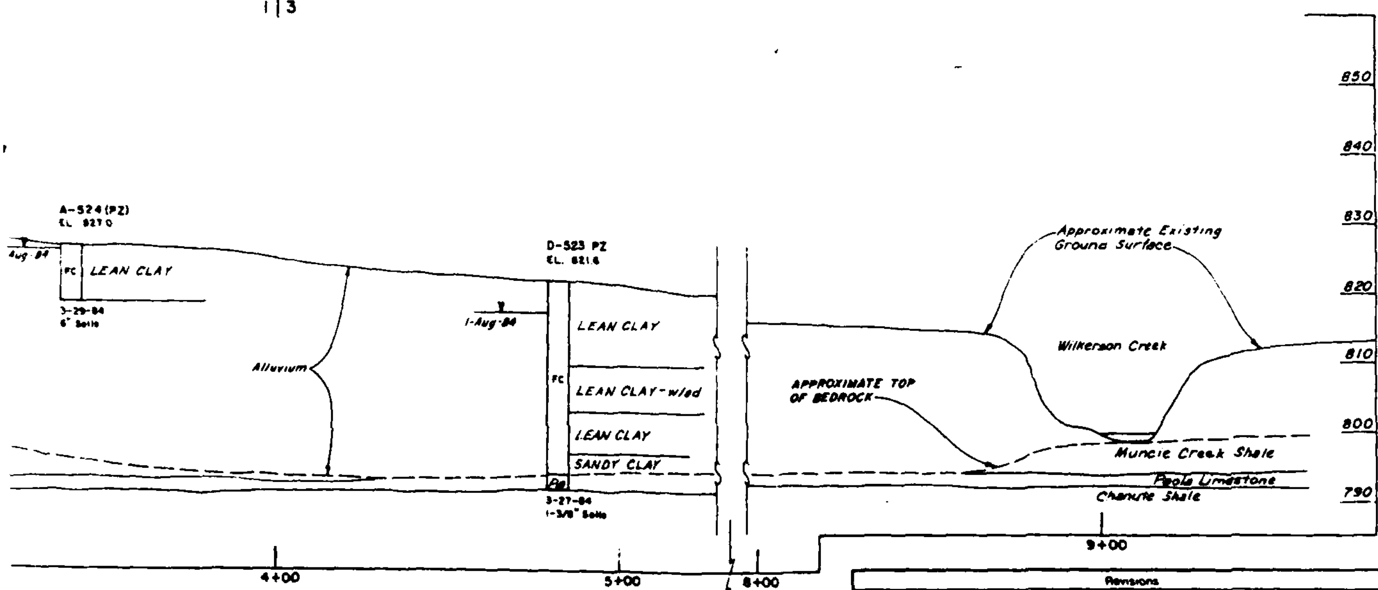


Notes:
1. Boundary between glacial drift and alluvium is not defined.
2. Piezometric surface shown is measured in the pervious basal layer of the overburden.

15-1-1



THROUGH LEFT ABUTMENT A 1|3



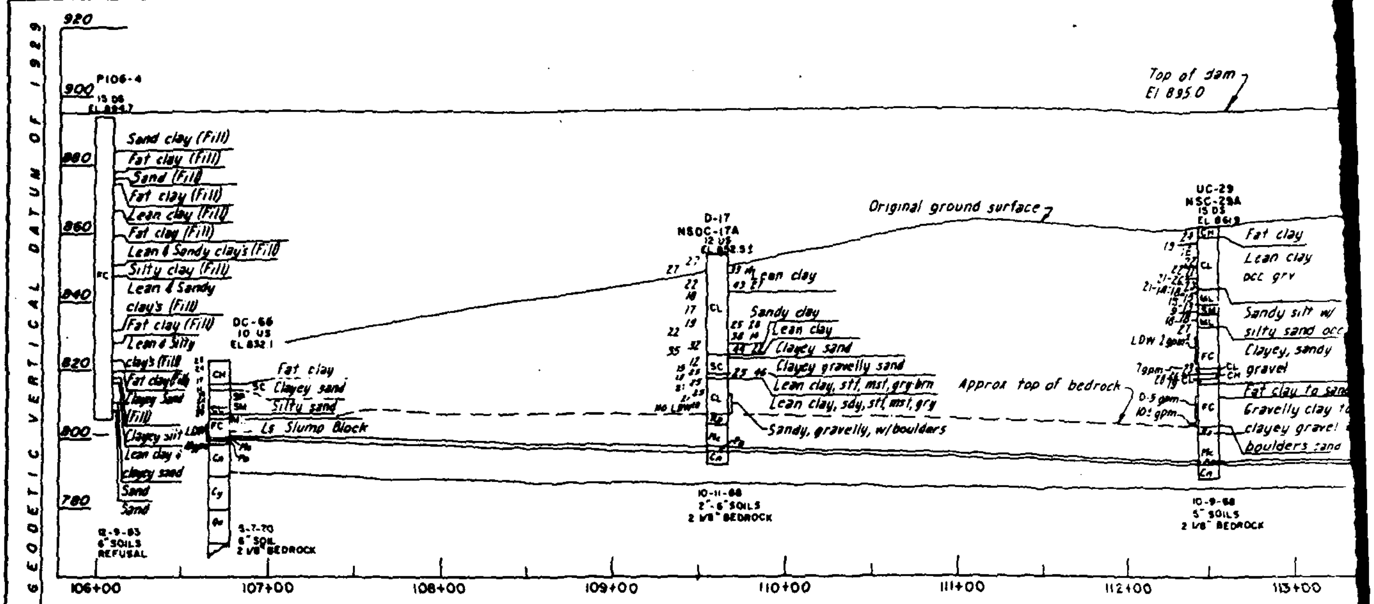
ROUGH BOWERS SEEP AREA B 1|3

Revisions			
Symbol	Descriptions	Date	Approved

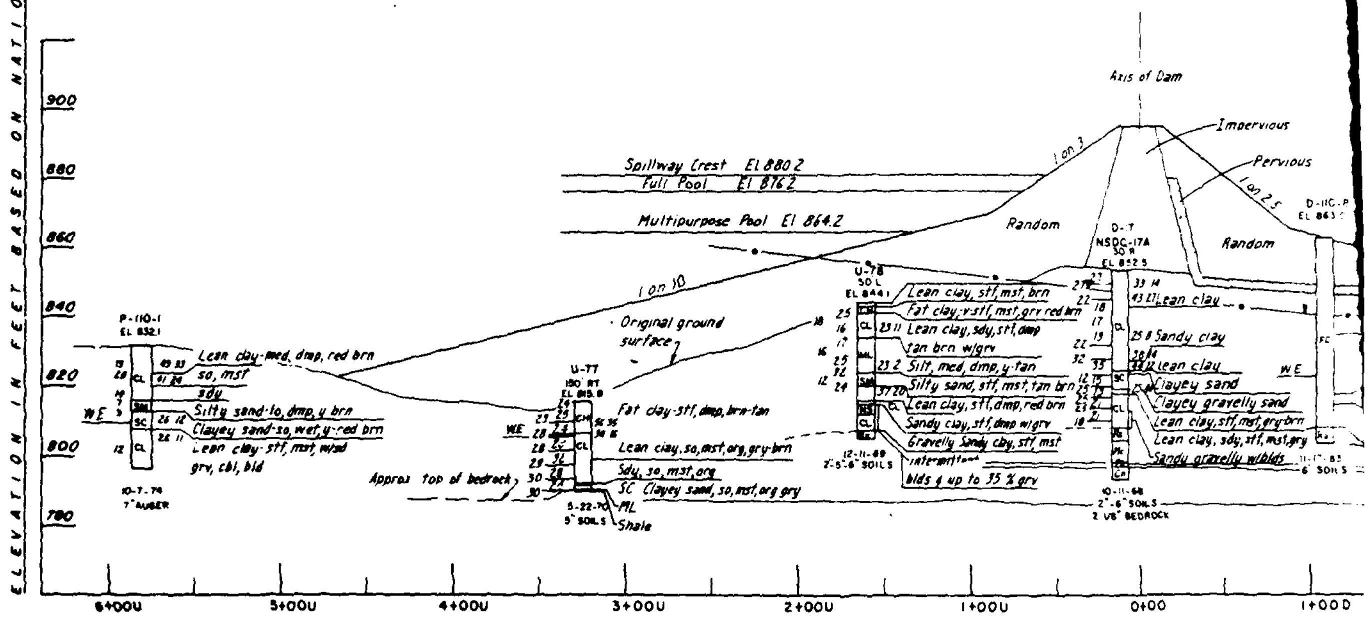
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J.M.M.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE LEFT ABUTMENT SEEPAGE REPORT
Drawn by: J.G.S.	PROFILES-LEFT ABUTMENT; BOWERS SEEP AREA
Checked by: H.L.F.	Scale: AS SHOWN — Sheet number
Submitted by: K.D.W.	Date: DECEMBER 1984 — RP-3-1723
	Dwg. No. — File No. [redacted]

APPENDIX B PLATE NO 3



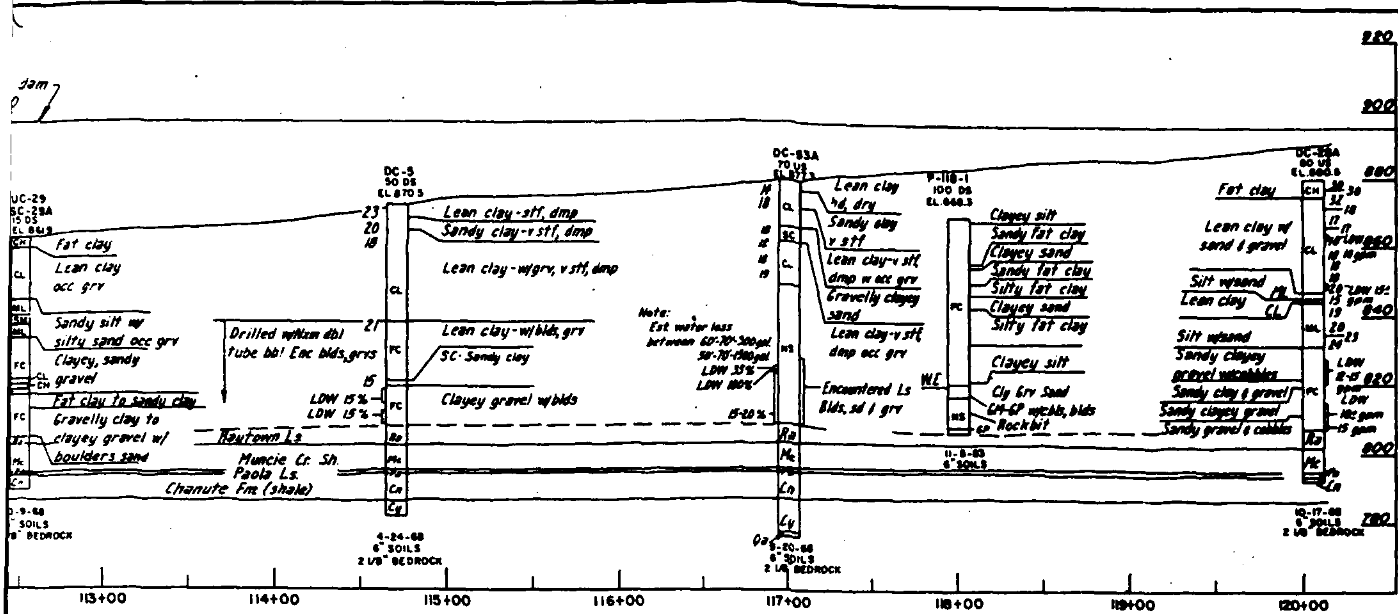
LEFT ABUTMENT DAM AX



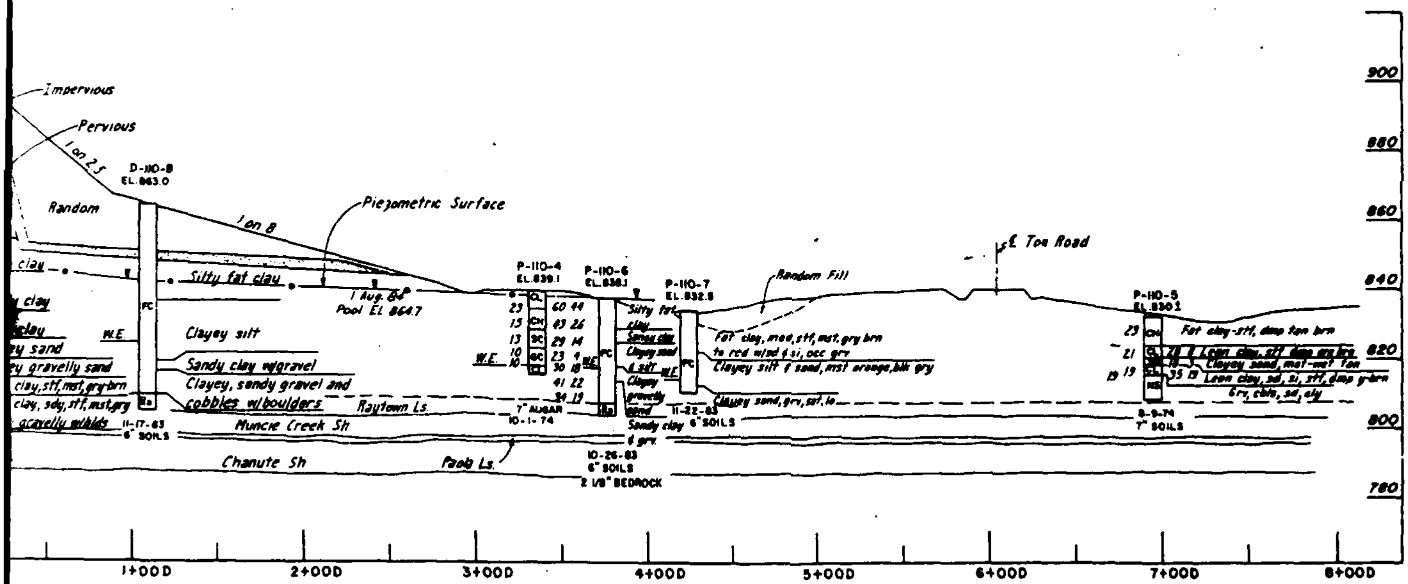
LEFT ABUTMENT
SECTION-STA. 110 + 00 (C / 1 | 4)

Note
1. Piezometric surface shown is measured in the pervious basal layer of the overburden.

15-1-7



BUTMENT DAM AXIS PROFILE



TMENT
0 + 00 C 1|4

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

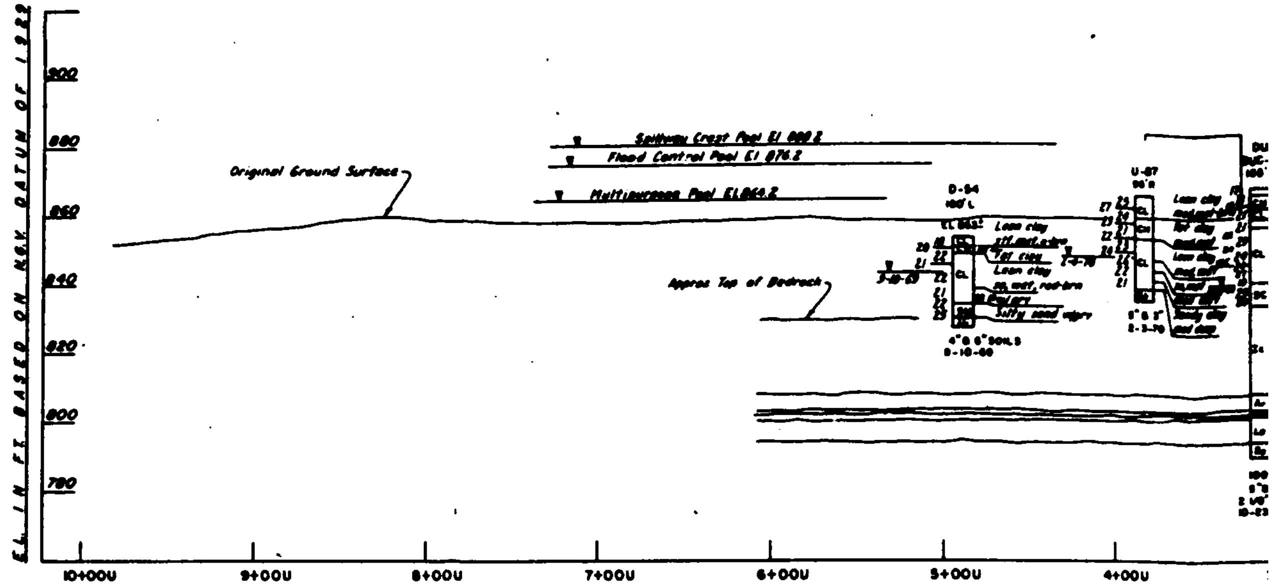
Designed by: W.F.L.
Drawn by: EER
Checked by: H.L.F
Submitted by: K.D.W.

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
LEFT ABUTMENT SEEPAGE REPORT

LEFT ABUTMENT DAM AXIS PROFILE;
EMBANKMENT SECTION-STATION 110 +00

Scale: AS SHOWN
Date: DECEMBER 1984
Sheet number:
RP-3-1724

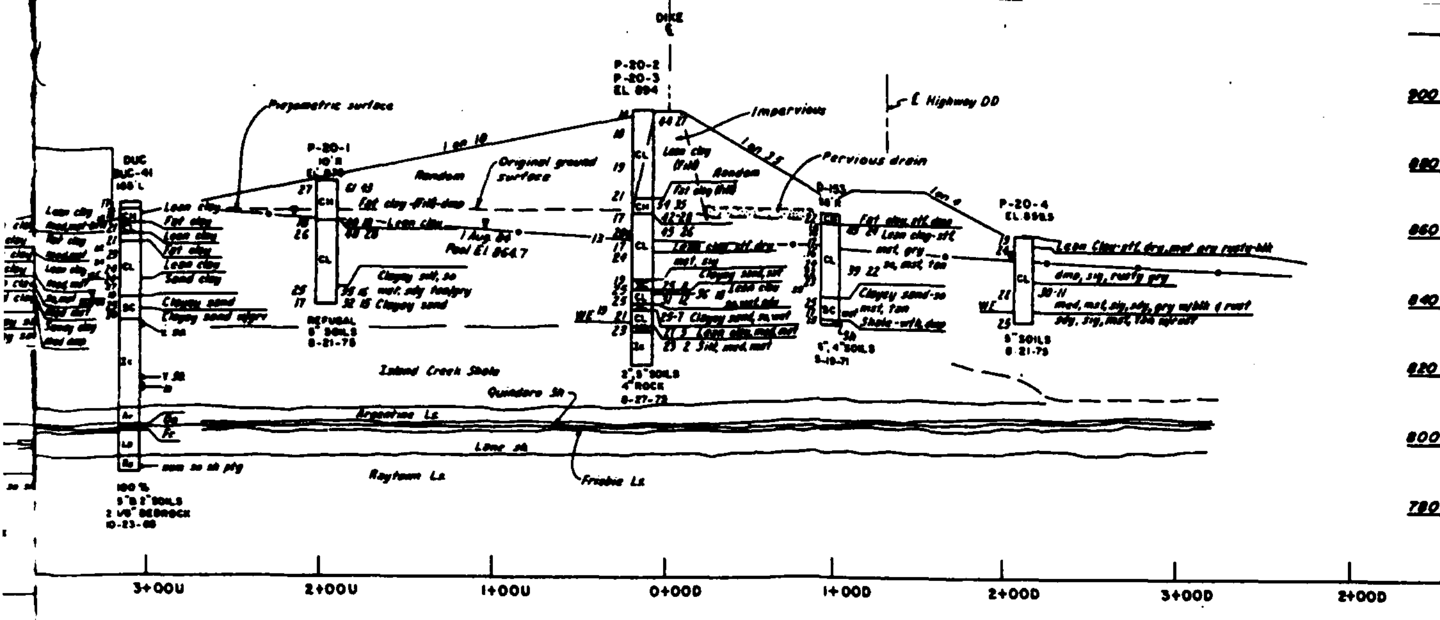
APPENDIX 8 PLATE NO. 4



MAIN DIKE

Note:
1. Piezometric surface show is measured in the pervious basal layer of the overburden.

15-1-7



MAIN DIKE SECTION D 115

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: W.F.L.
Drawn by: E.E.R.
Checked by: H.L.F.
Submitted by: K.D.W.

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
LEFT ABUTMENT SEEPAGE REPORT

MAIN DIKE SECTION-STATION 20+00

Scale: AS SHOWN
Date: DECEMBER 1984
Sheet number:
RP-3-1725
File No. [redacted]

APPENDIX B PLATE NO. 5

1

2

GEOLOGIC COLUMN - LEFT ABUTMENT, SMITHVILLE DAM

OVERBURDEN variable thickness, lean clays, Silty clays w/occasional lenses of fine sand & occasional fine to coarse gravel, underlain by clayey silts & fine to medium sands w/ clay & occasional gravel. Clayey, sandy gravels w/ limestone chert & quartzite cobbles & boulders.

WYANDOTTE FM - ISLAND CREEK SHALE 26'
Soft, platy, sl clayey, calcareous, few phosphatic nodules in upper 15', lt gray to gray. Soft to v soft, clayey, w/ high angle fractures (approximately 15 to 16' above base of Island Creek). Moderately hard to hard, dense, medium to thin bedded, brown-gray limestone w/ few small open vugs & occasional wavy shale parting (approximately 5 to 7' above base of Island Creek).

WYANDOTTE FM - ARGENTINE LIMESTONE 5'
Moderately hard to hard, dense, thin wavy bedded, brown-gray, w/ numerous soft dk gray wavy shale partings & bands.

WYANDOTTE FM - QUINDARO SHALE 0.5'
Soft, very calcareous, gray, fossiliferous.

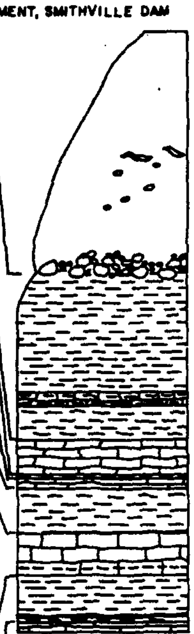
WYANDOTTE FM - FRISBIE LIMESTONE 1'
Moderately hard to hard, dense, thick-bedded, tabular & fossiliferous inclusions, gray-brown.

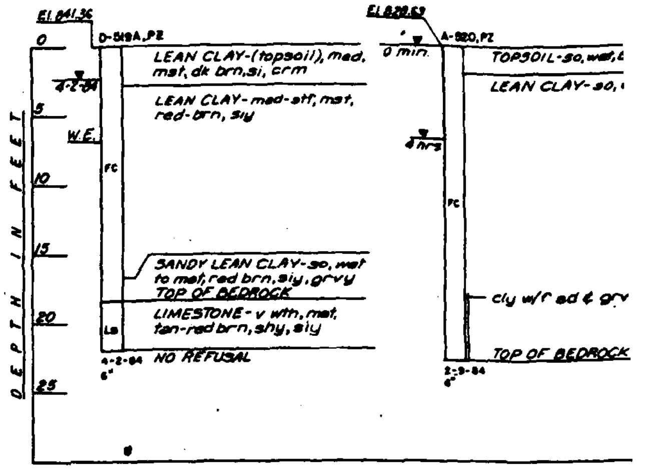
LANE FM - SHALE average thickness 7'
Soft, fissile, slightly calcareous, dark gray.

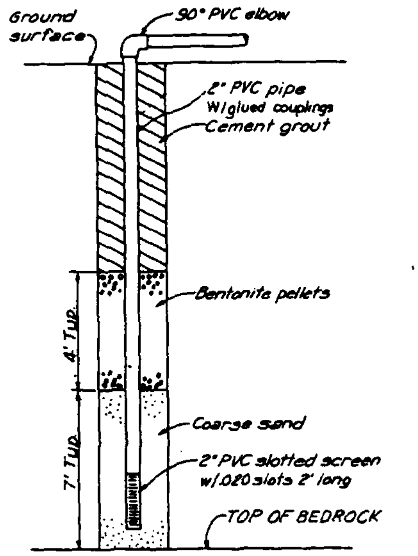
IOLA FM - RAYTOWN LIMESTONE 6'
Moderately hard to hard, crystalline to dense fossiliferous & massive in upper part. Thin-bedded w/ interlaminated soft, calcareous shale w/ persistent soft to v soft clayey shale (approximately 1 to 1.5' above base of Raytown) overlying moderately hard, dense to finely crystalline fossiliferous limestone w/ shaly partings in lower part.

IOLA FM - MUNCIE CREEK SHALE 7'
Soft to moderately hard w/ occasional v soft thin laminae, platy, dk gray to black, calcareous, coal to black carbonaceous sh near base.

IOLA FM - PAOLA LIMESTONE 1'
Moderately hard to hard, finely crystalline, massive w/ occasional argillaceous partings, gray fossiliferous.

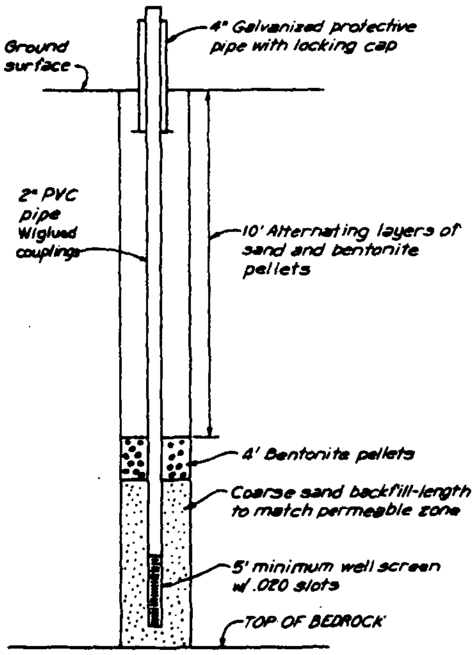




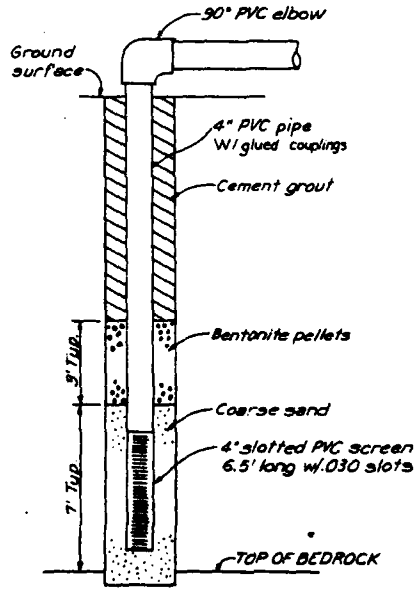


TYPICAL 2-INCH RELIEF DRAIN INSTALLATION

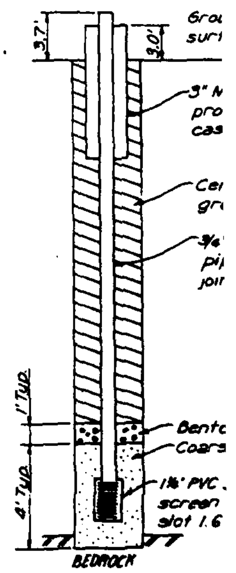
alt	alternating	dmp	
ang	angular	dol (c)	
an	anhydrite	ext	
ar	argillaceous	f (y)	
bdd	bed, bedded, bedding	fe	
bdr	bedrock	fld	
bky	blocky	fm	
bl	blue	fos (s)	
bld	boulder	frac (d)	
blk	black	frag (d)	
brec (d)	breccia, (brecciated)	fri	
brk	broken	fsl	
brn	brown	gr	
c	coarse	grs	
calc	calcareous	grn	
carb	carbonaceous	grv (y)	
cav	cavity	gry	
cbl	cobble	gyp	
cht	chert	hs	
circl	circulation	hd	
cl (y)	clay, (clayey)	hld	
cld	closed	hor	
cmtd	cemented	inbd	
col	columnar	incl	
conc	concretions	inlam	
cong	conglomerate	irr	
crm	crumbly	jt (s)	
d	dense	le	
dk	dark	lam (d)	



TYPICAL 2-INCH INSTALLATION
OBSERVATION WELL

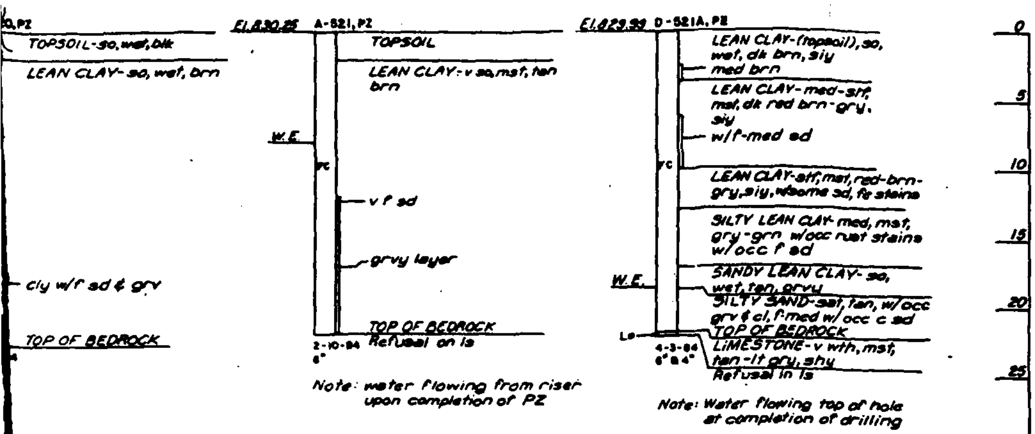


TYPICAL 4-INCH RELIEF DRAIN INSTALLATION



TYPICAL 3/4-INCH PIEZ
INSTALLATION

15-1-7



UNIFIED SOIL CLASSIFICATION SYSTEM

GW	Well graded gravels, gravel-sand mixtures, little or no fines.	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts w/slight plasticity.
GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
GM	Silty gravels, gravel-sand-silt mixtures.	OL	Organic silts and organic silty clays of low plasticity.
GC	Clayey gravels, gravel-sand-clay mixtures.	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
SW	Well-graded sands, gravelly sands, little or no fines.	CH	Inorganic clays of high plasticity, fat clays.
SP	Poorly graded sands or gravelly sands, little or no fines.	OH	Organic clays of medium to high plasticity, organic silts.
SM	Silty sands, sand-silt mixtures.	Pt	Peat and other highly organic soils.
SC	Clayey sands, sand-clay mixtures.		

Classification from actual laboratory tests where LL and PI are shown.
Dual classification, where used, is in accordance with the Unified Soil Classification System.
For details on the Unified Soil Classification System, See Waterways Experiment Station Technical Memorandum No. 3-357 dated March 1953 and revised in 1960.

DETACHED BORINGS

ABBREVIATIONS

dmp	damp	lea	leached	rnd (d)	round, (rounded)
dol (c)	dolomite, (dolomitic)	lig	lignite	sat	saturated
ext	extremely	ls	limestone	scat	scattered
f (y)	fine, (finely)	lt	light	sd (y)	sand, (sandy)
fe	iron	lo	loose	sev	several
fld	filled	L.C.	lost core	sh (y)	shale, (shaly)
fm	firm	L.D.W.	lost drill water	si (y)	silt, (silty)
fos (s)	fossil, (fossiliferous)	med	medium	sis	siltstone
frac (d)	fractures, (fractured)	mic	micaceous	sl	slightly
frag (d)	fragments, (fragmented)	min	mineralized	slcs	siliceous
fri	friable	mod (y)	moderate, (moderately)	slks	slickensides
fsl	fissile	mot	mottled	so	soft
gr	grain	mas	massive	sol (d)	solution, (solutionized)
gra	gradation	mst	moist	ss	sandstone
grn	green	mtl	material	st (g)	stained, (staining)
grv (y)	gravel, (gravelly)	mtx	matrix	stf	stiff
gry	gray	nod	nodules	sty	stylolitic
gyp	gypsum	num	numerous	v	very
ha	high angle	occ (y)	occasional, (occasionally)	vert	vertical
hd	hard	op	open	vgy	vuggy
hld	healed	or	orange	w	water
hor	horizontal	org	organic	w/	with
inbd	interbedded	par	partially	wth	weathered
incl	inclusions	pit	pit, pitted, pitting	wht	white
inlam	interlaminated	pl	plastic	x-bdd	crossed-bedded
irr	irregular	pla	platy	xln	crystalline
jt (s)	joint, (joints)	pln	plane	y	yellow
la	low angle	ptg (s)	parting, (partings)	When used as log symbol first letter is capitalized	
lam (d)	laminae, (laminated)	qtz (e)	quartz, (quartzite)		

TERMS FOR CONSISTENCY OF SOIL AND HARDNESS OF BEDROCK

SOIL

Consistency	Estimated Unconfined Compressive Strength (Tons per square foot)
Very soft	< 0.25
Soft	0.25—0.5
Medium	0.5—1.0
Stiff	1.0—2.0
Very stiff	2.0—4.0
Hard	> 4.0

BEDROCK

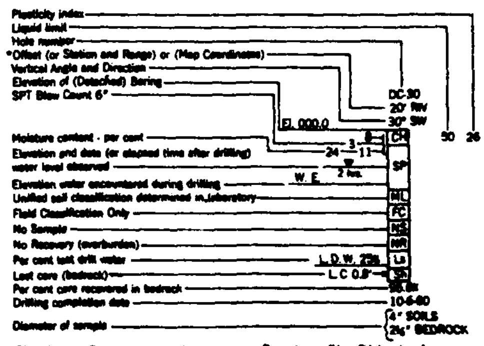
SCALE OF HARDNESS

Very soft or plastic	Can be indented easily with thumb.
Soft	Can be scratched with fingernail.
Moderately hard	Can be scratched easily with knife; cannot be scratched with fingernail.
Hard	Difficult to scratch with knife.
Very Hard	Cannot be scratched with knife.

BEDROCK UNIT THICKNESS

Parting	< 0.02'
Band	0.02' to 0.2'
Thin Bed	0.2' to 0.5'
Medium bed	0.5' to 1.0'
Thick bed	1.0' to 2.0'
Massive	> 2.0'

LEGEND FOR LOGS OF BORINGS



*Offset from profile or section may be Upstream or Downstream (U or D.) Landward or Riverward (LWD or RIV), or Right or Left (R or L) as defined

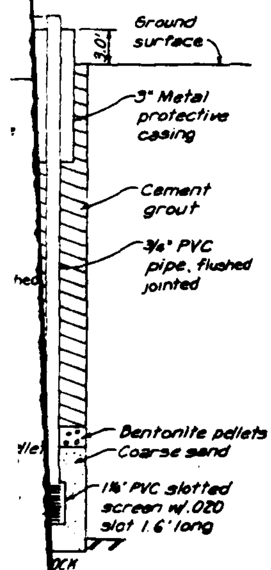
TYPE OF EXPLORATION

MAP SYMBOL

- Vertical boring
- 30° Inclined boring showing direction and vertical angle

CODE DESIGNATION

- D Drive sample hole
- C Core hole
- TP Test pit (includes power auger 24" or larger diameter)
- U Undisturbed sample hole
- A Auger hole-hand or power auger less than 24" diameter.
- NS Not Sampled (Field Classification from cuttings only)
- FS Field Section of outcrop.
- OW Observation well
- RD Relief drain
- P Piezometer no.
- PZ Piezometer



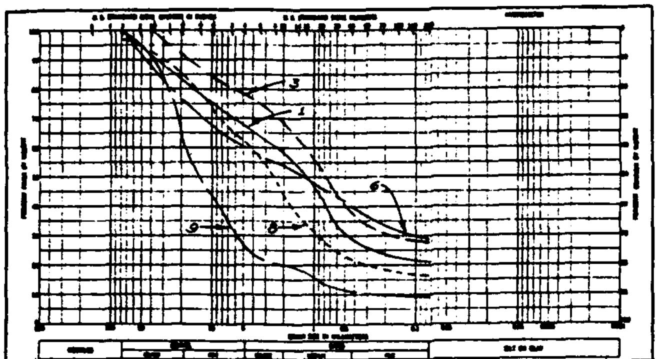
/4-INCH PIEZOMETER INSTALLATION

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by:	J.M.M.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE LEFT ABUTMENT SEEPAGE REPORT
Drawn by:	R.L.D.	GEOLOGIC COLUMN AND LEGEND, DETACHED BORINGS, AND TYPICAL INSTALLATION DIAGRAMS
Checked by:	H.L.F.	Scale: AS SHOWN — Sheet number:
Submitted by:	K.D.W.	Date: DECEMBER 1984 — Dwg. No.: — RP-3-1726 — File No. [redacted]

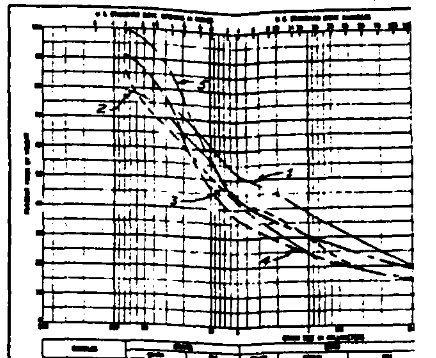
APPENDIX B PLATE NO. 6



1	26.2-26.0	SM						Smithville Lake
3	28.1-28.0	SC						Sack Samples
6	32.0-34.0	GM						
8	36.0-40.0	SM						D-513
9	60.0-62.0	GM-GP						21 Sept 83

GRADATION CURVES

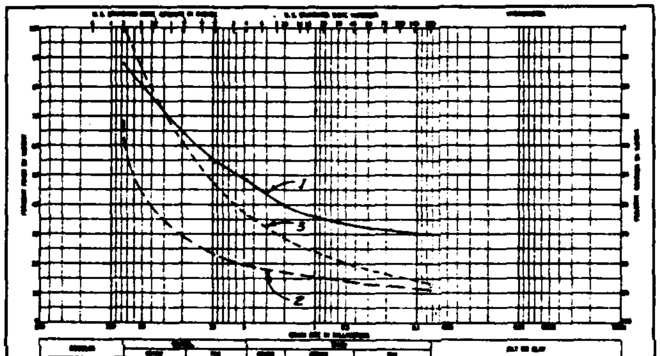
ENG FORM 2087



1	43.0-44.3	GM				
2	50.0-51.8	GM				
3	58.0-60.0	GM				
4	60.0-62.0	GM				
5	64.5-66.0	GC				

GRADATION CURVES

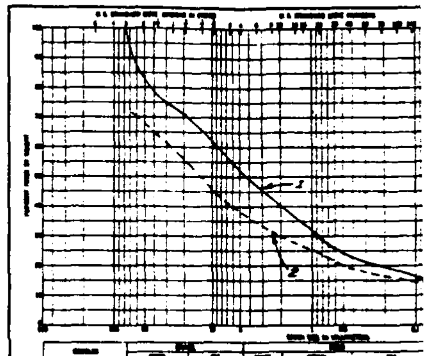
ENG FORM 2087



1	53.0-54.5	GM						Smithville Lake
2	52.5-60.0	GM-GP						Sack Samples
3	65.0-66.0	GM						
								D-517
								7 Oct 93

GRADATION CURVES

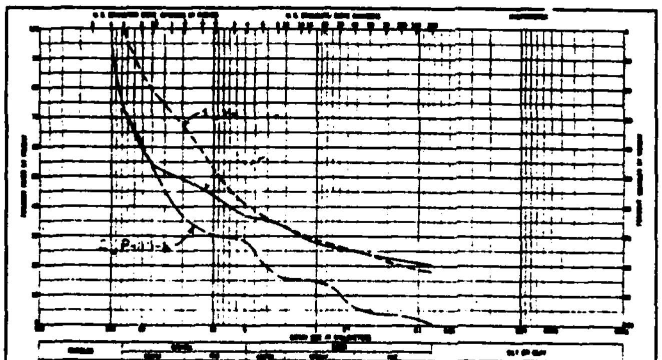
ENG FORM 2087



1	62.0-66.0	GM				
2	74.0-76.0	GC				

GRADATION CURVES

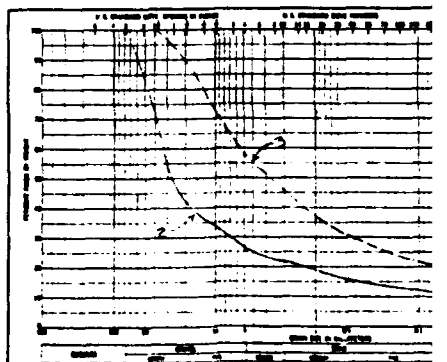
ENG FORM 2087



1	25.0-26.0	GM						Smithville Lake
2	27.0-28.0	GM						Sack Samples
	P-110-B							
2	50.5-51.5	GW						P-110-6, P-110-B
								26 Oct 83 / 11 ... 83

GRADATION CURVES

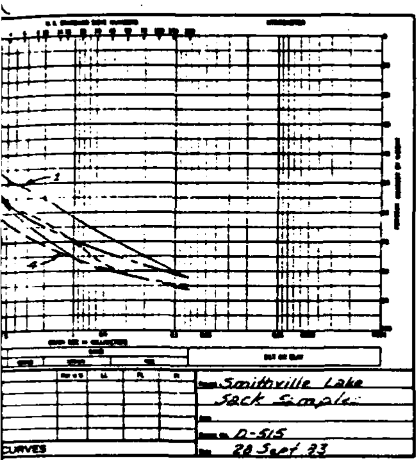
ENG FORM 2087

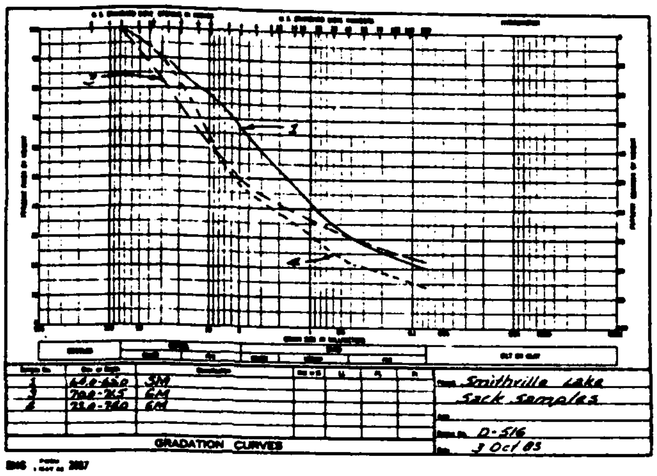


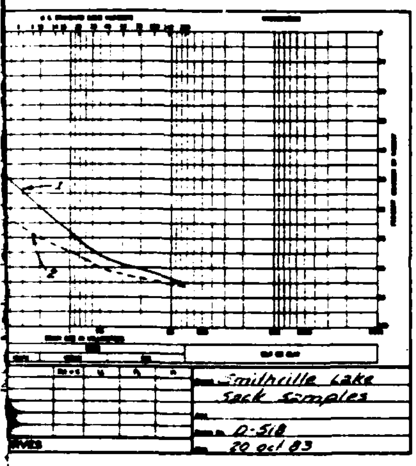
GRADATION CURVES

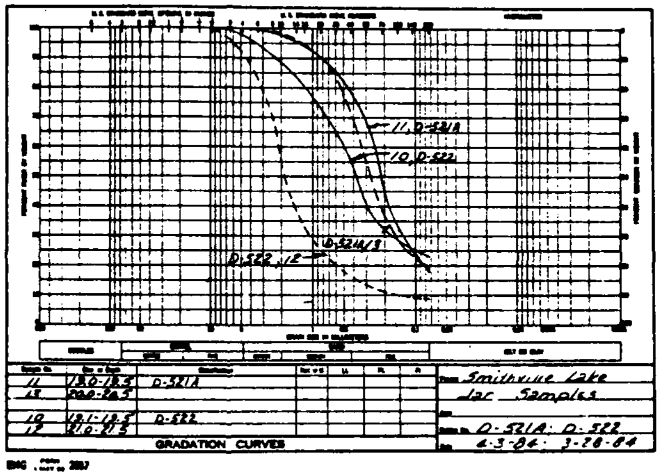
ENG FORM 2087

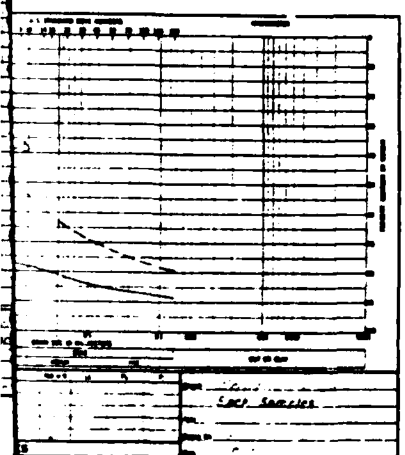
-1-7









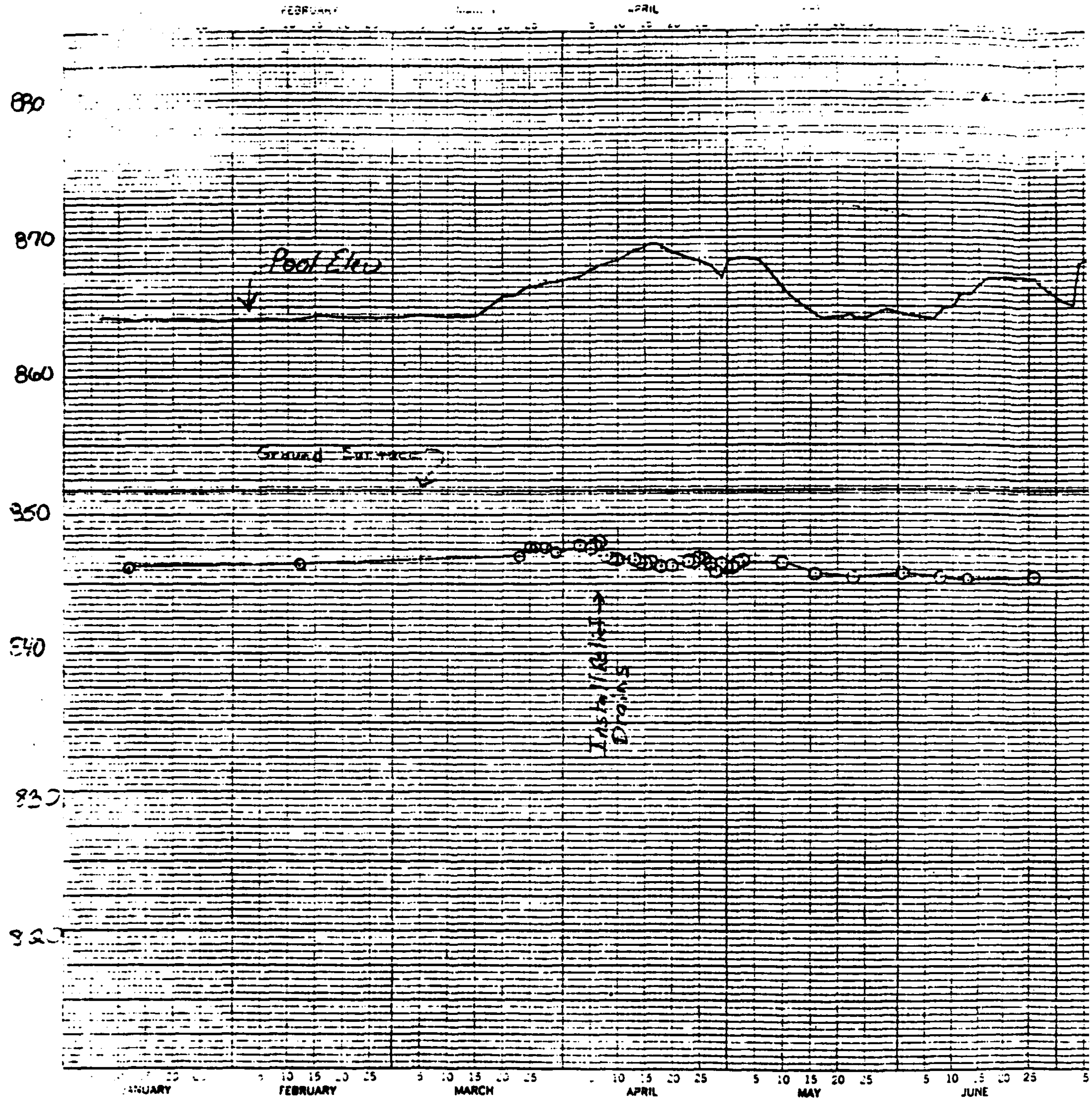


Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J.M.M	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE LEFT ABUTMENT SEEPAGE REPORT		
Drawn by: P.S.T.	GRADATION CURVES LEFT ABUTMENT BORINGS		
Checked by: H.L.F	Scale: AS SHOWN	Sheet number:	RP-3-1727
Submitted by: K.D.W.	Date: DECEMBER 1984		File No. [redacted]
	Dwg. No.:		

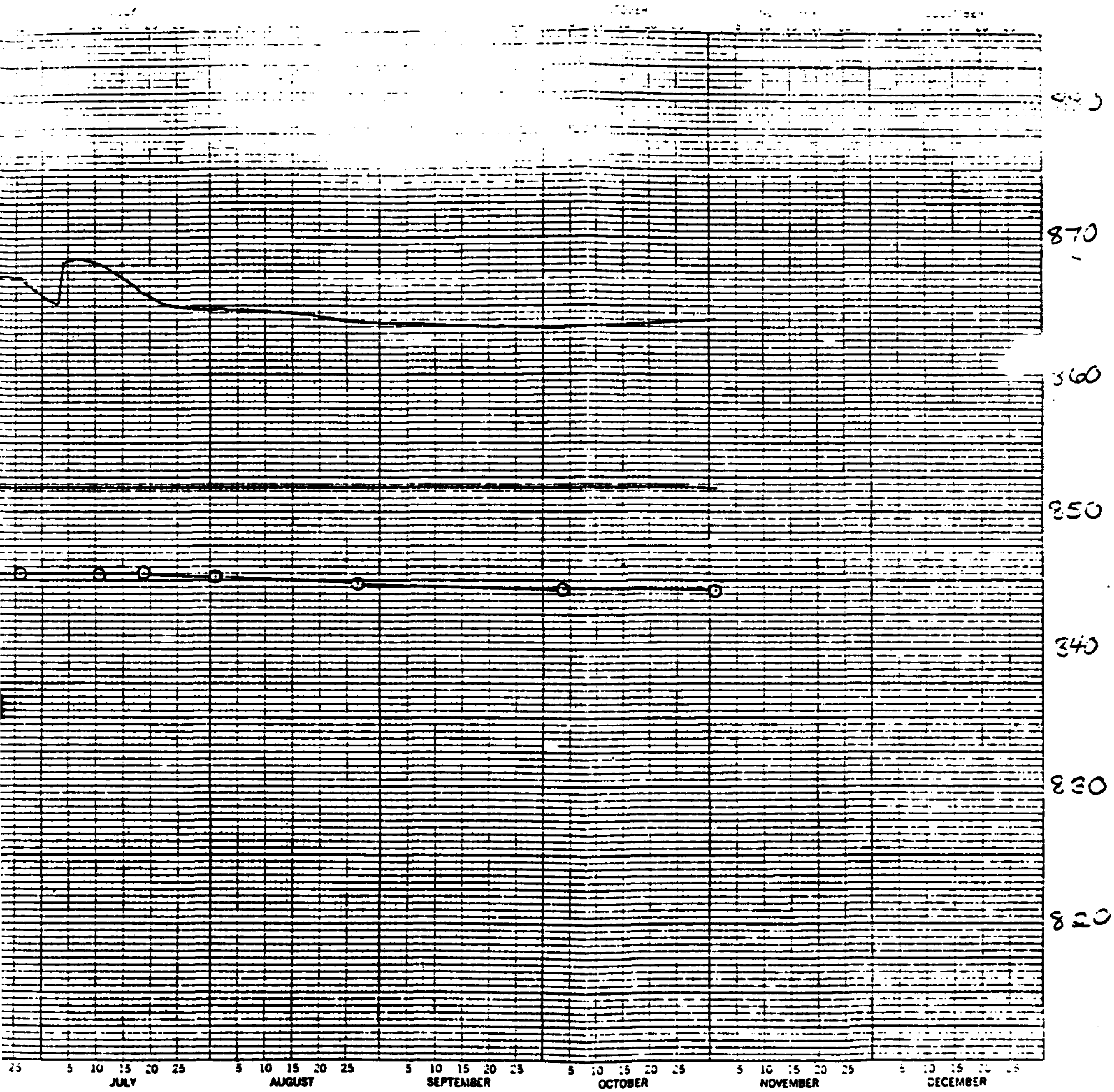
APPENDIX B PLATE NO. 7



880
870
860
850
840
830
820
Pool Elev
Ground Surface
Install Relief Drains
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE

1984

15-1-7

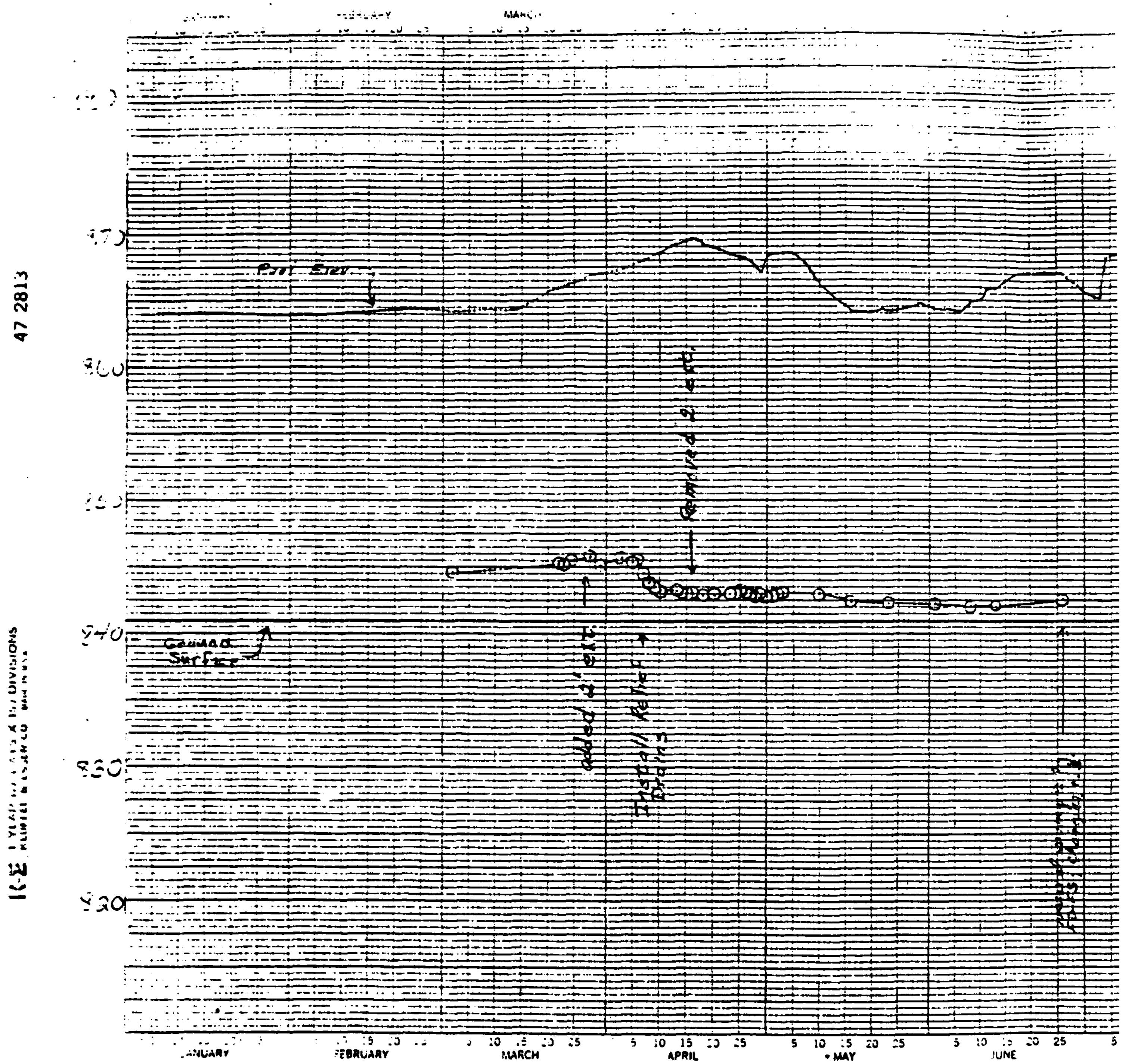


1984

D-513

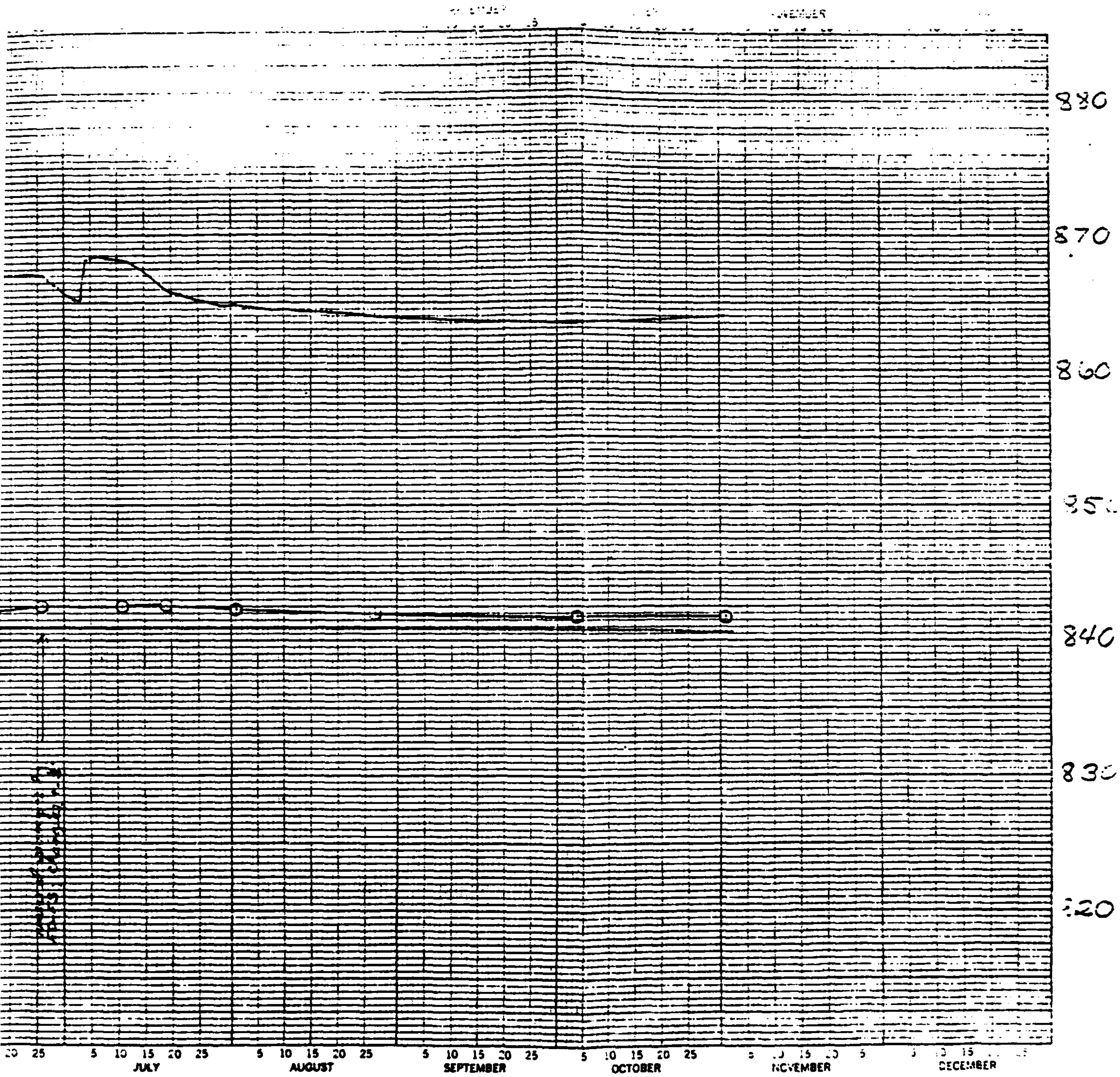
STA TIP EL 810.0
RNG GRND EL 851.8
MAT'L CL SDY GRV

APPENDIX B Plate No. 8 RP-3-1728


Pzt Elev
Ground Surface
Added 2' ext
Install Relief Drains
Removed 2' ext
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
47 2813
K-E

1984

15-1-7



1984

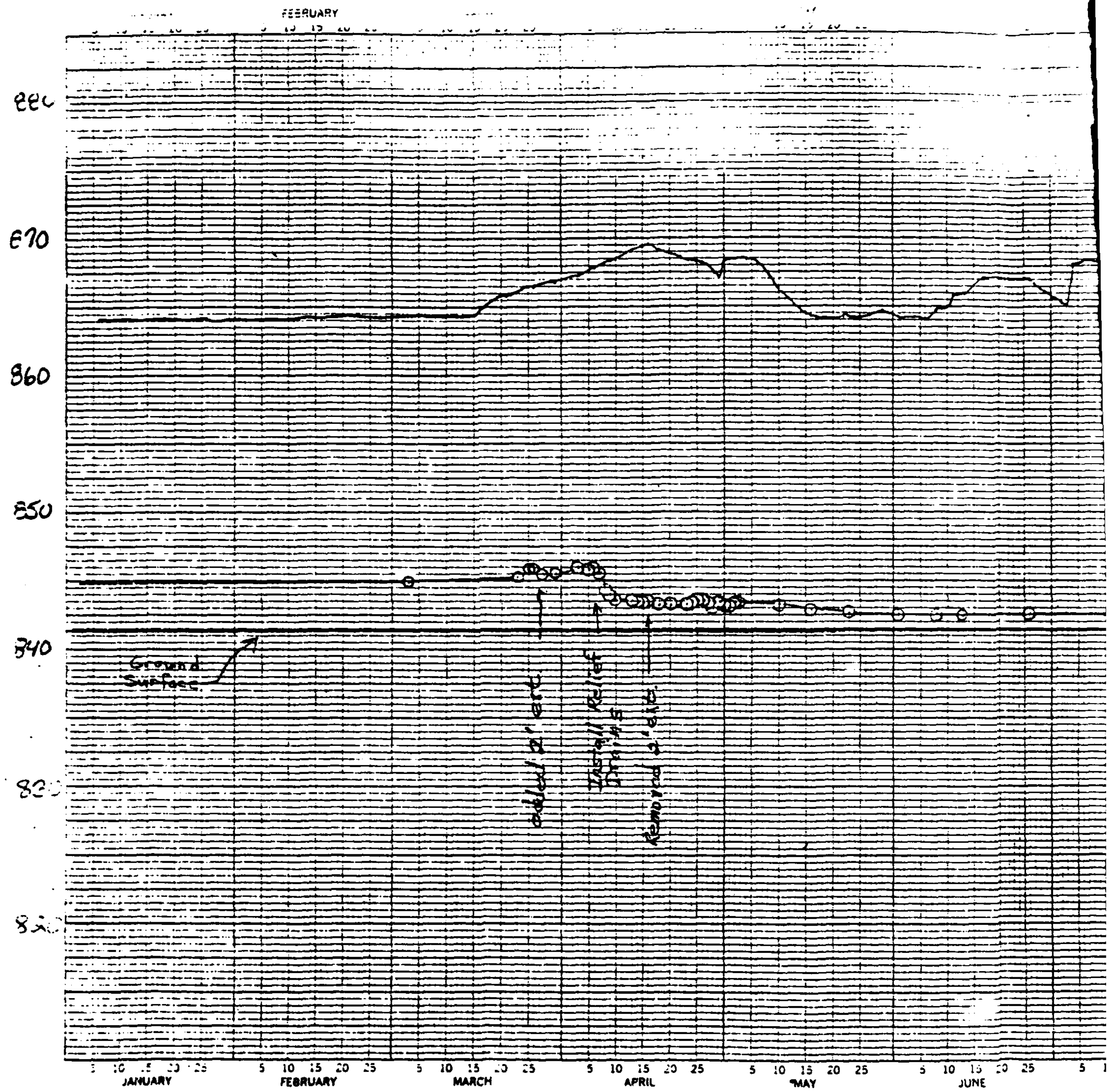
D-5[illegible]

STA TIP EL 817.7

RNG GRND EL 841.0

MAT'L CLY SAND

Plate No. 9 RP-3-1729

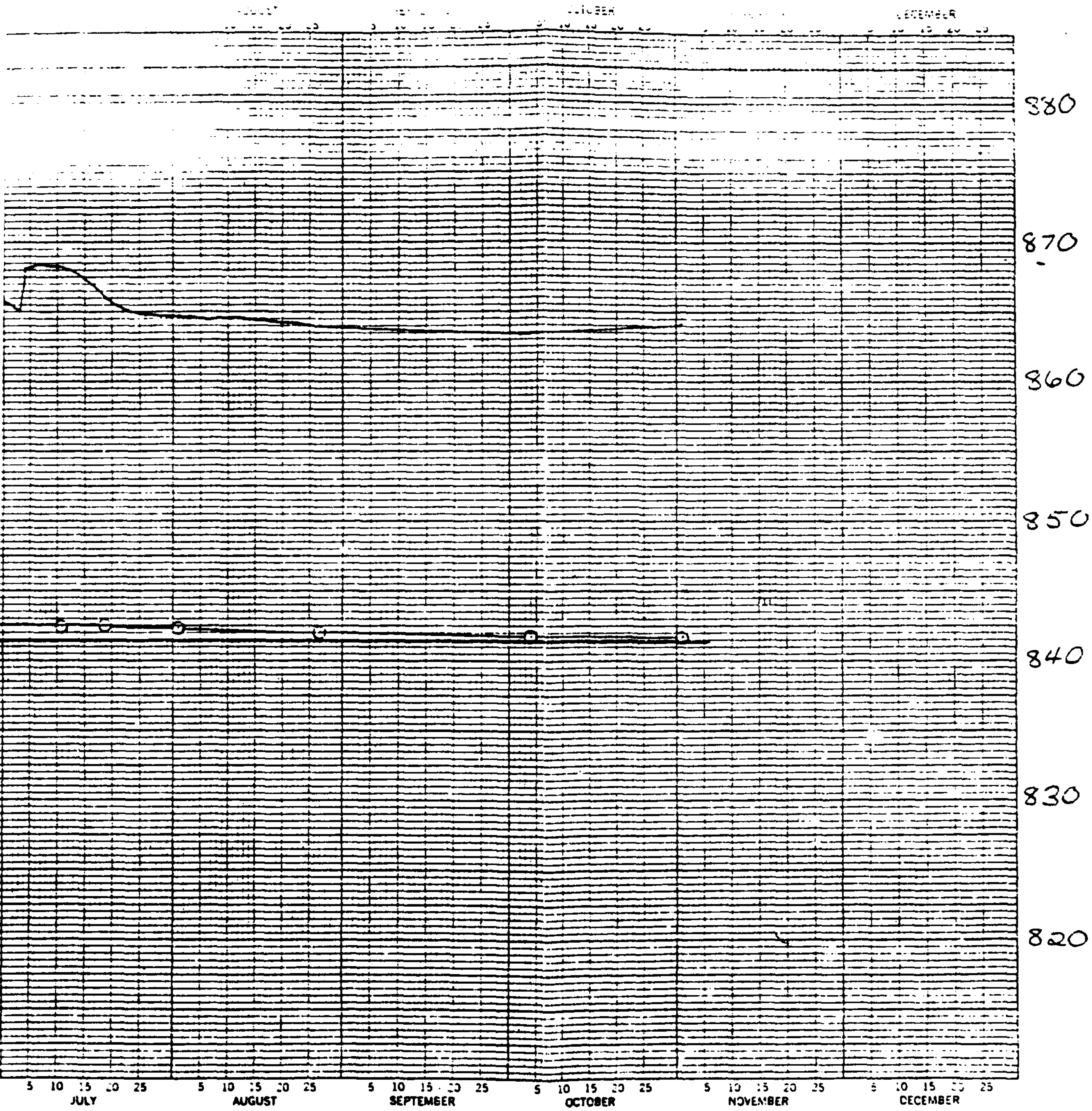


K+E 1 YEAR BY DAYS X 150 DIVISIONS KEUFFEL & ESSER CO. MADE IN U.S.A.

47 2813

1984

15-1-7



34

DC-51EA

STA TIP EL 809.9

RNG GRND EL 841.4

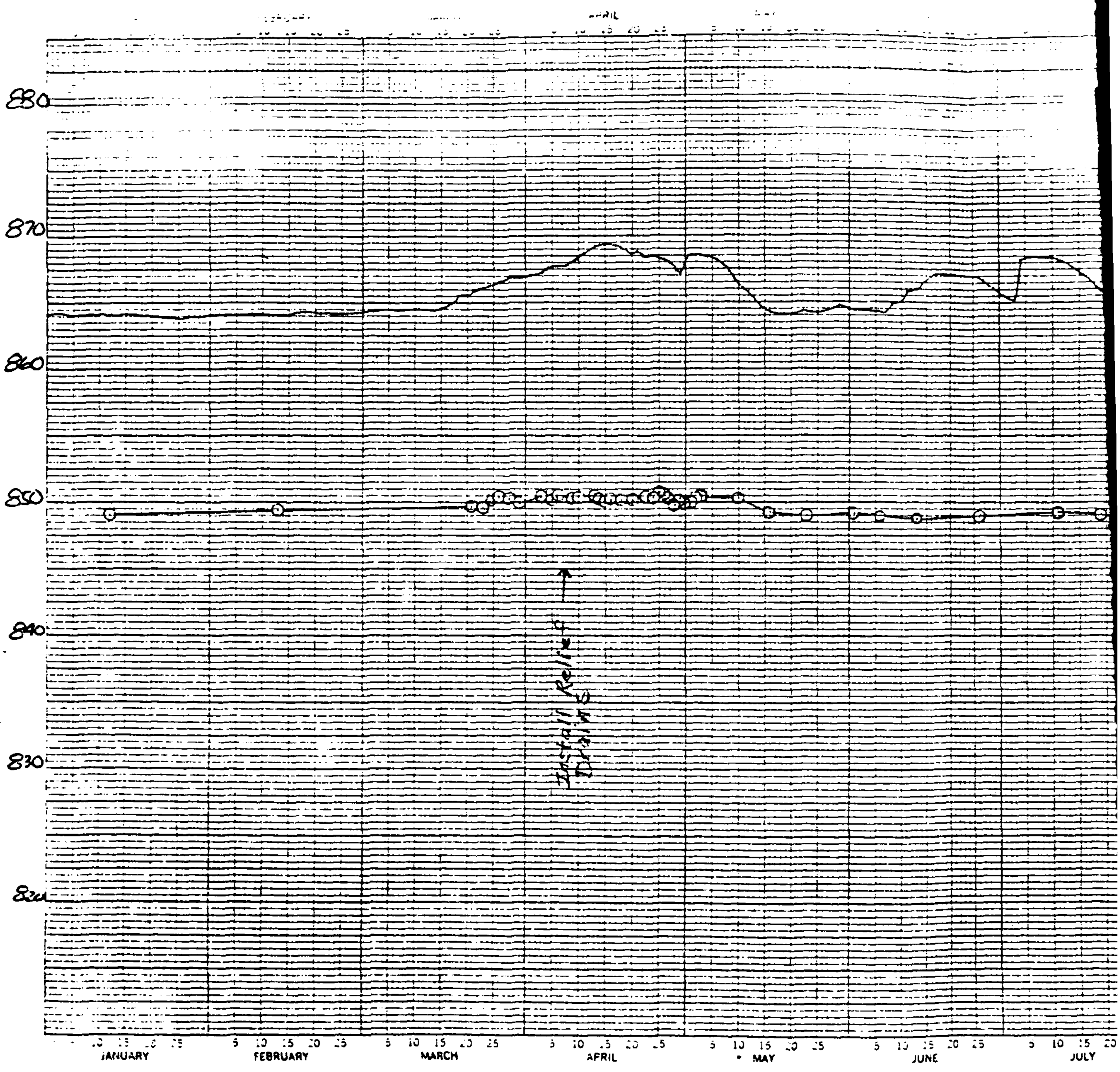
MAT'L CLY GRVY SD

APPENDIX B Plate No. 10 RP-3-1730

1

2

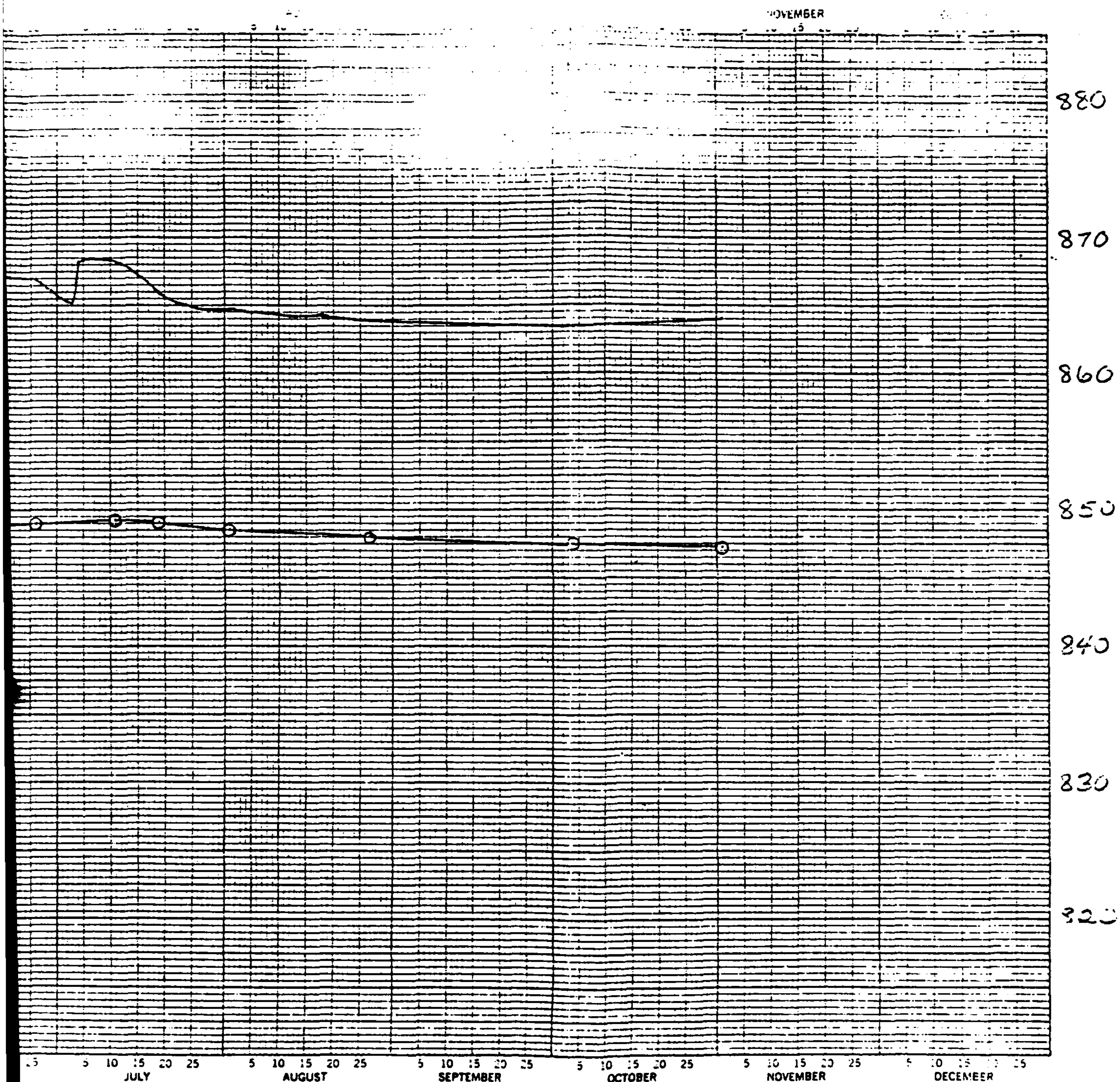
47 2813



K&E 1 YEAR BY DAYS X 150 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

1984

15-1-7



984

D-515

STA TIP EL 807.6

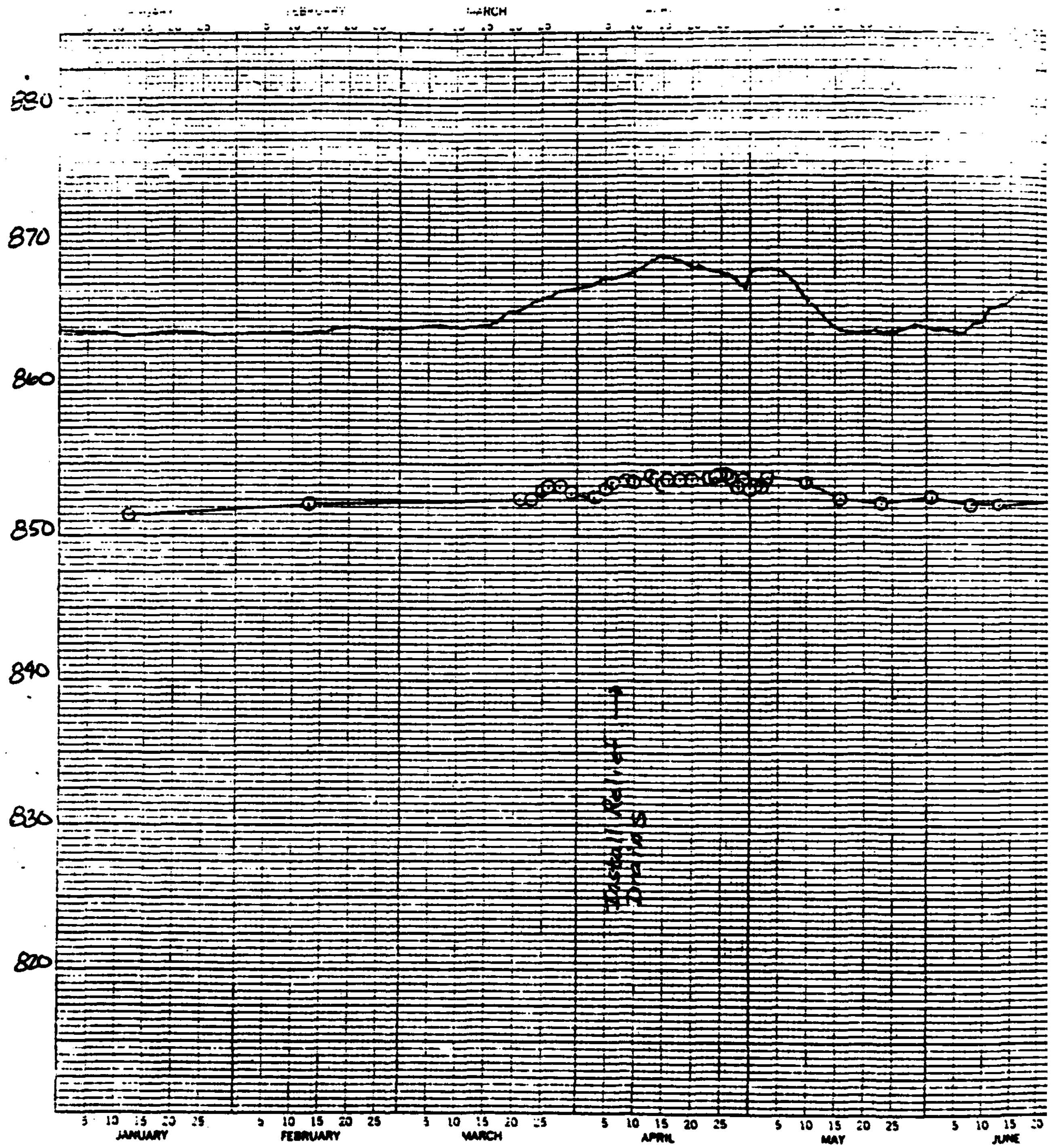
RNG GRND EL 877.1

MAT'L SIY GRV SD

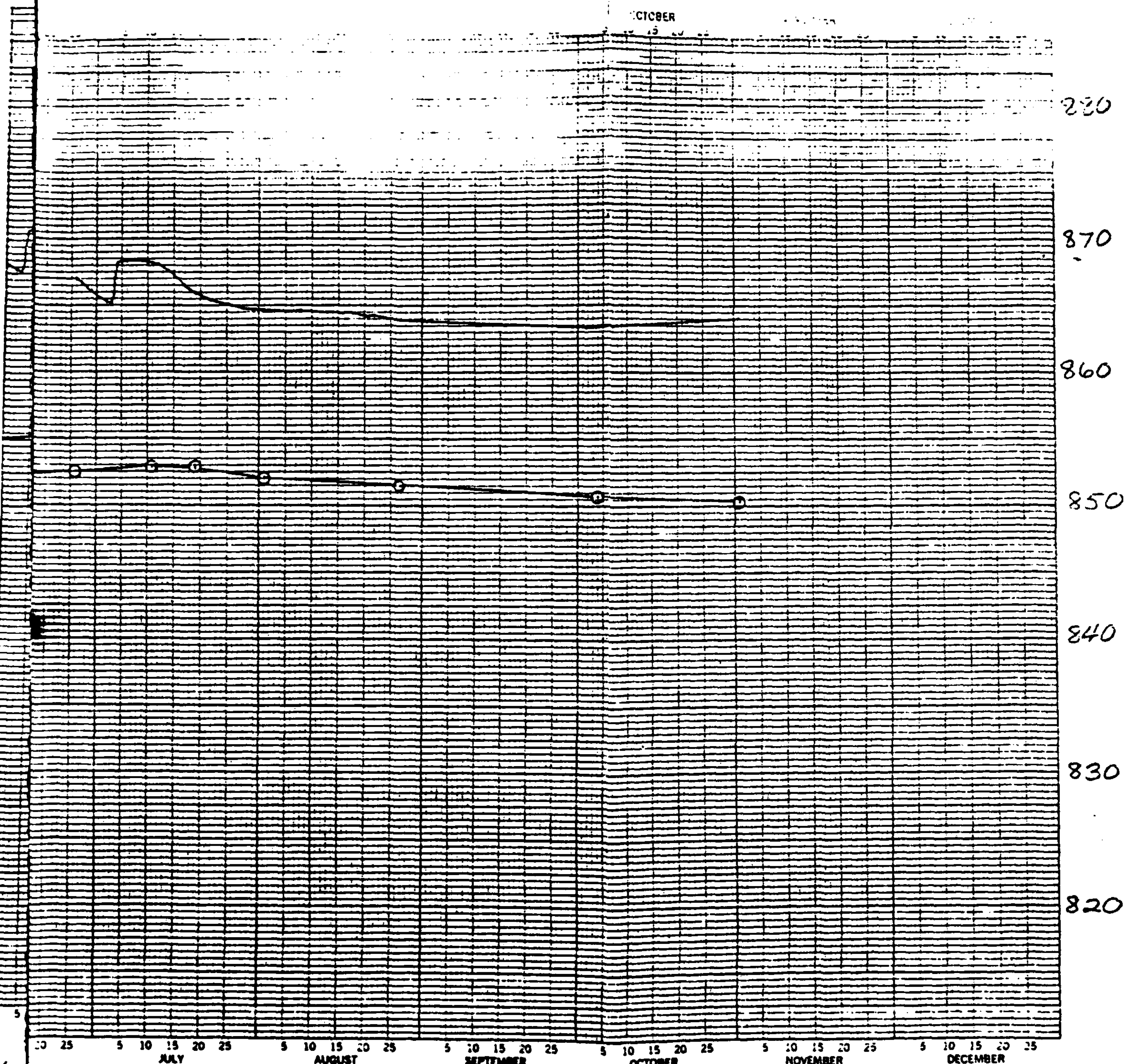
APPENDIX B Plate No. 11 RP-3-1731

K·E 1 YEAR BY DAYS X 150 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

47 2813



15-1-7

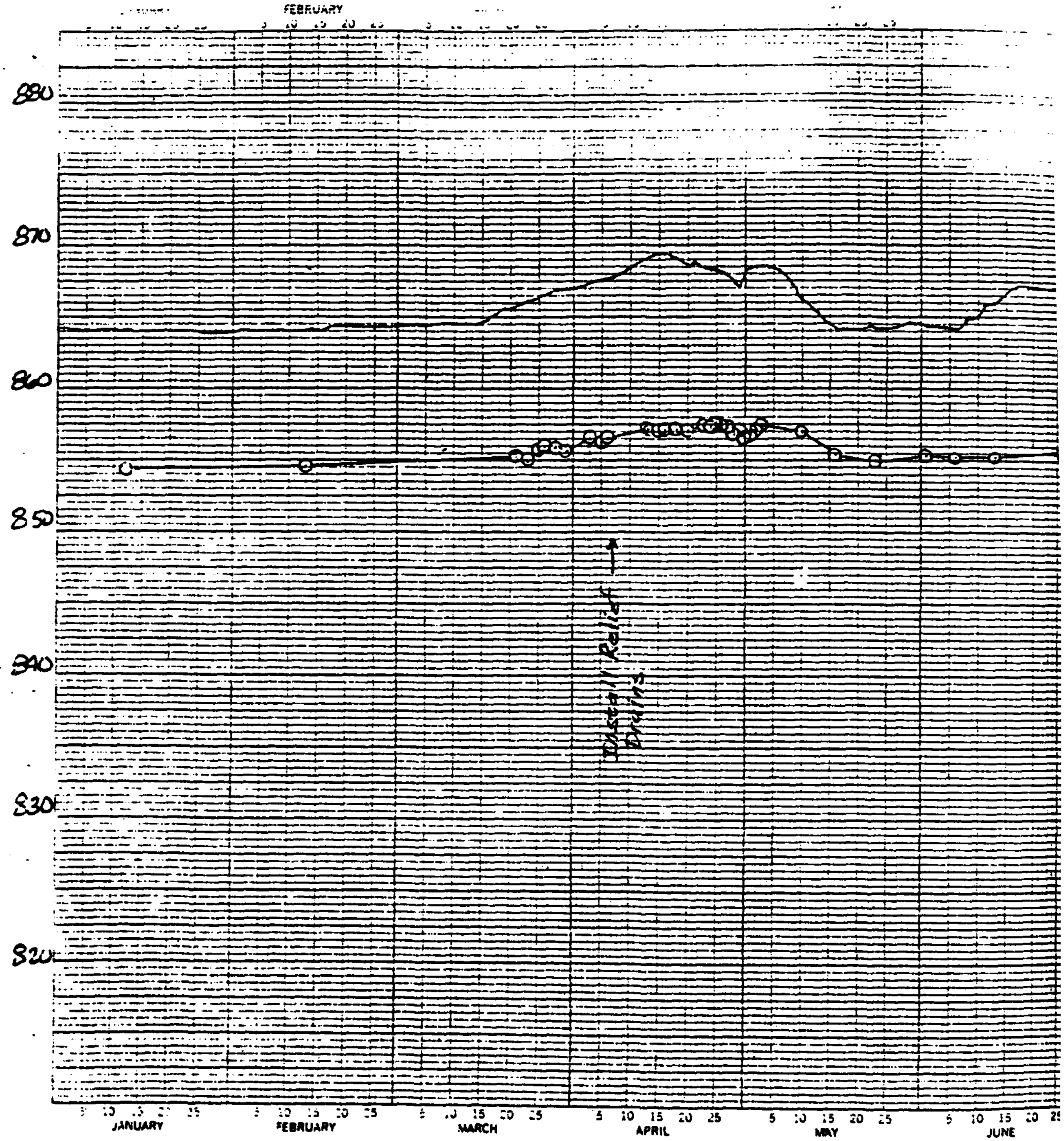


1984

D-516

STA TIP EL 812.7
RNG GRND EL 892.9
MAT'L SLY GRV SD

APPENDIX B Plate No. 12 RP-3-1732

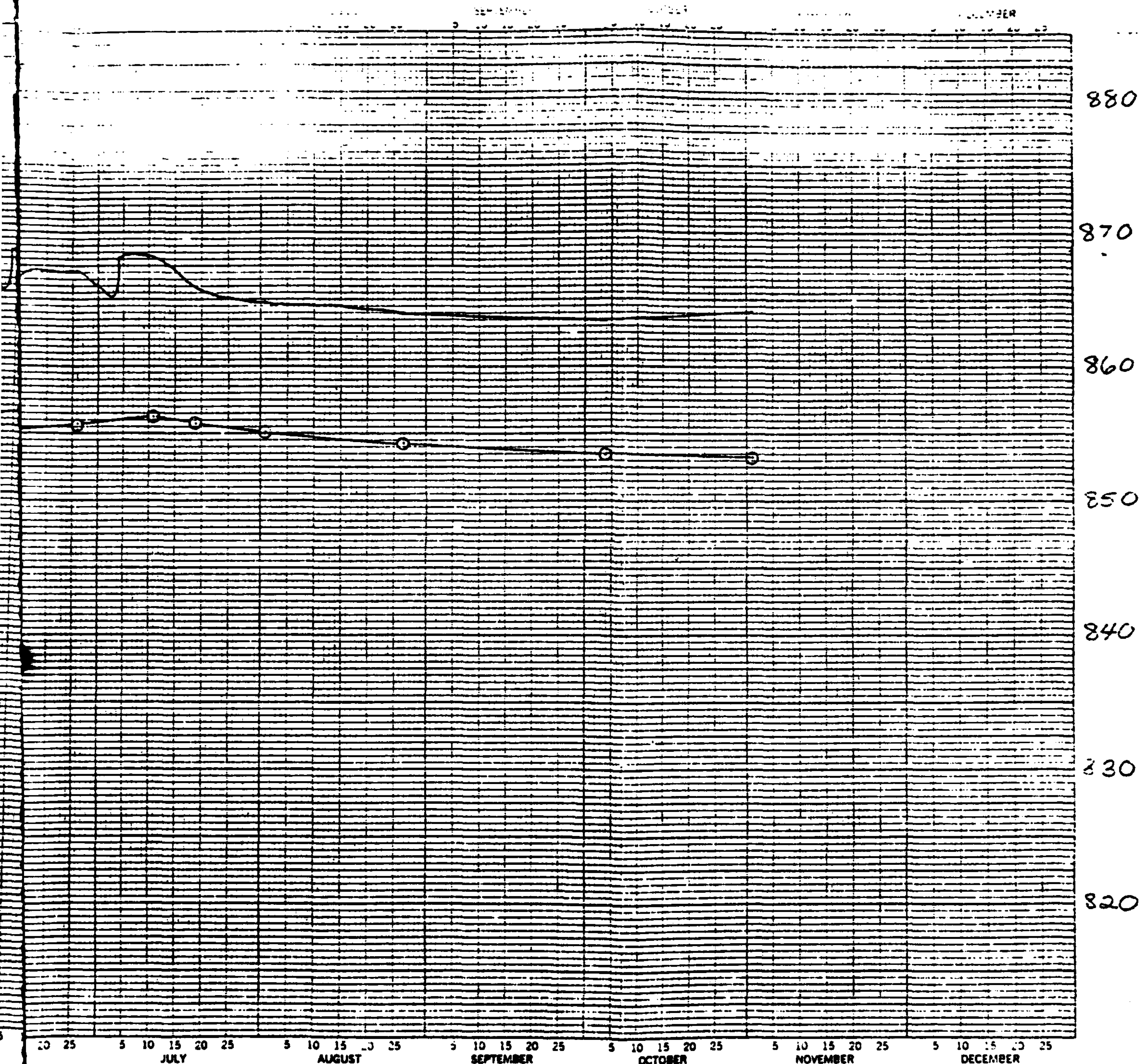


47 2813

19

15-1-7

1



1984

DC-517

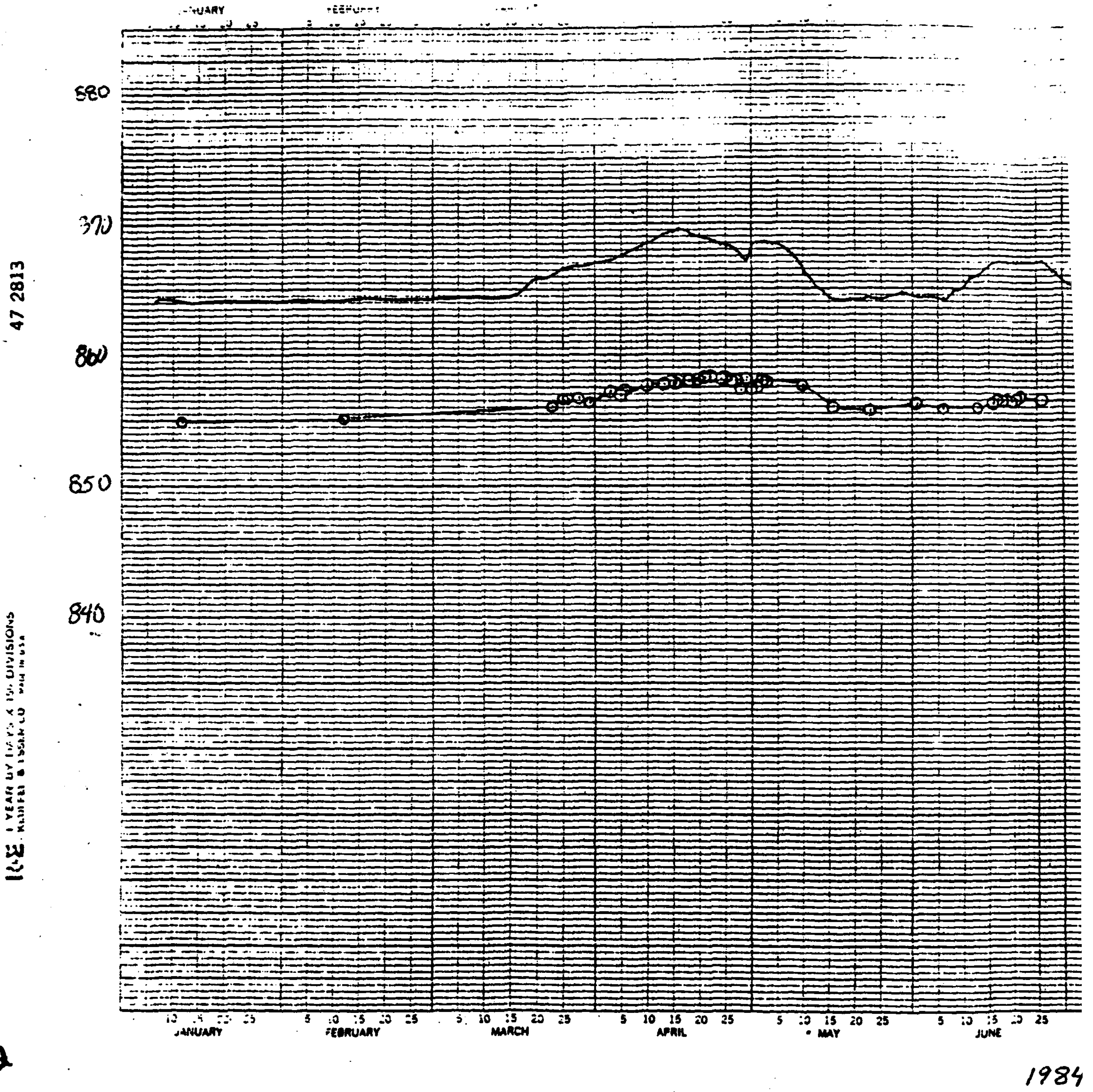
STA TIP EL 812.9

RNG GRND EL 880.0

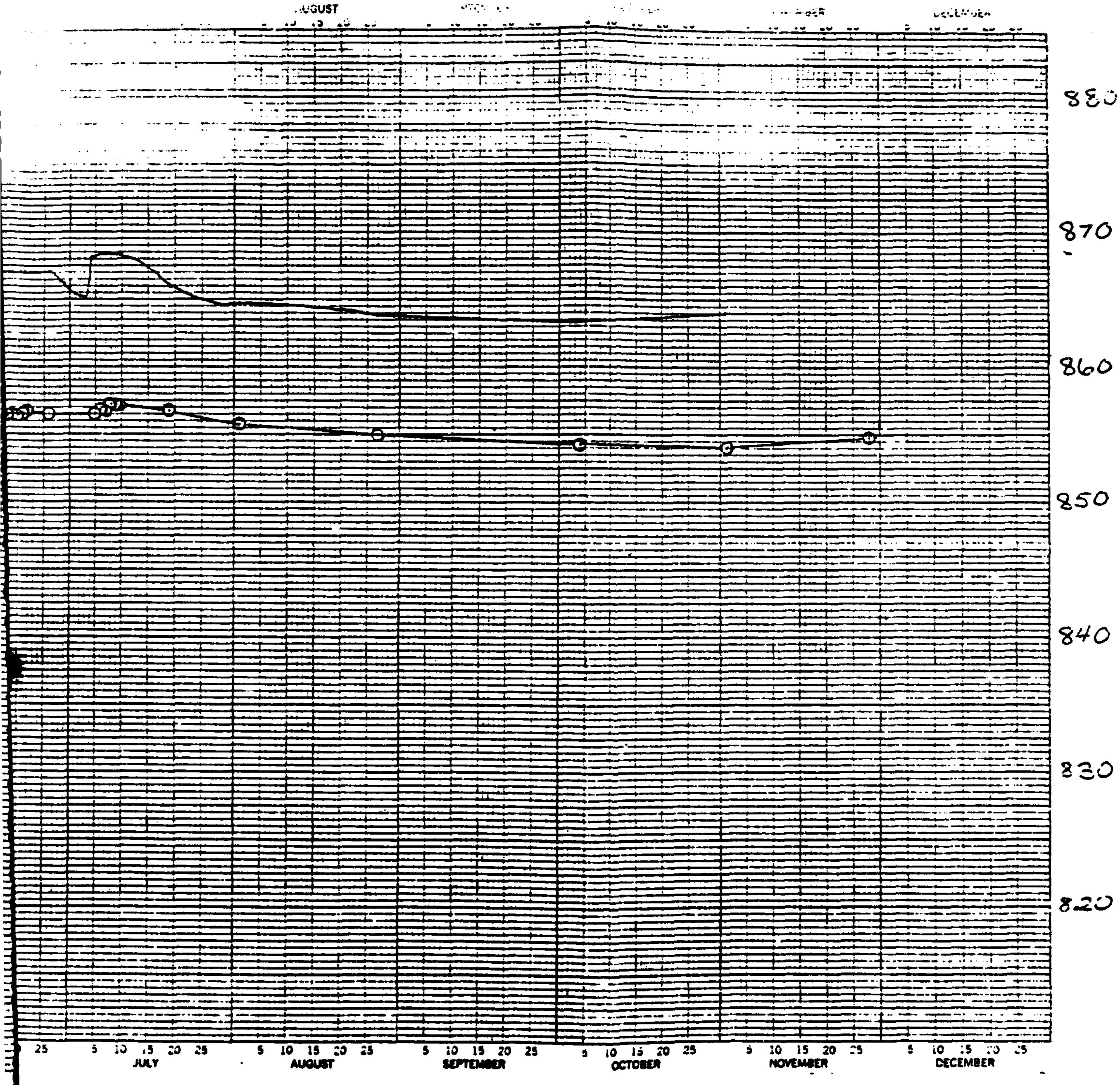
MAT'L SIY GRV SD

APPENDIX B Plate No. 13 RP-3-1733

2


880
870
860
850
840
10 15 20 25
JANUARY
5 10 15 20 25
FEBRUARY
5 10 15 20 25
MARCH
5 10 15 20 25
APRIL
5 10 15 20 25
MAY
5 10 15 20 25
JUNE
1984
47 2813

15-1-7

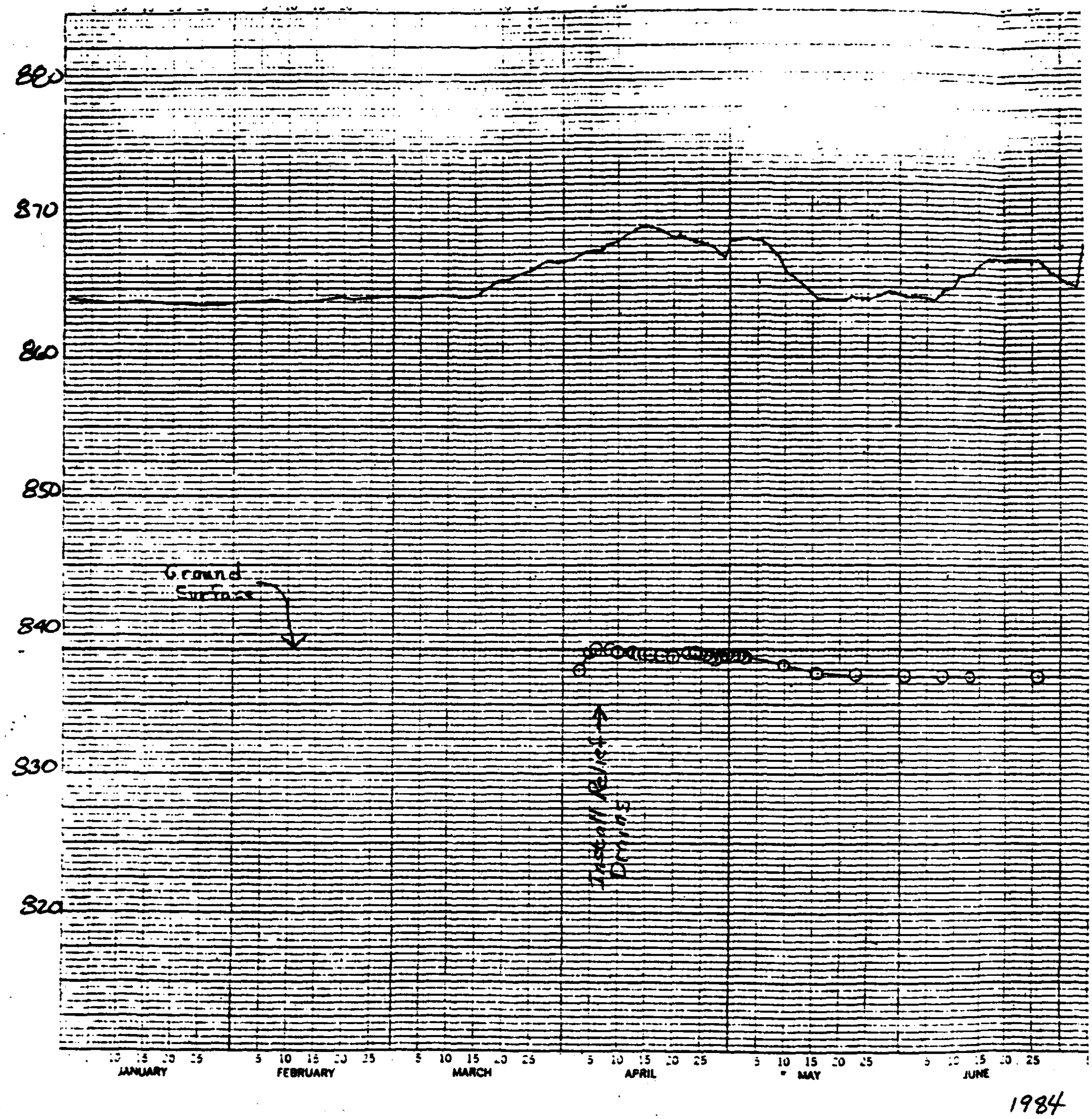


1984

D-518

STA TIP EL 823.3
RNG GRND EL 896.3
MAT'L SDY CLY GRV

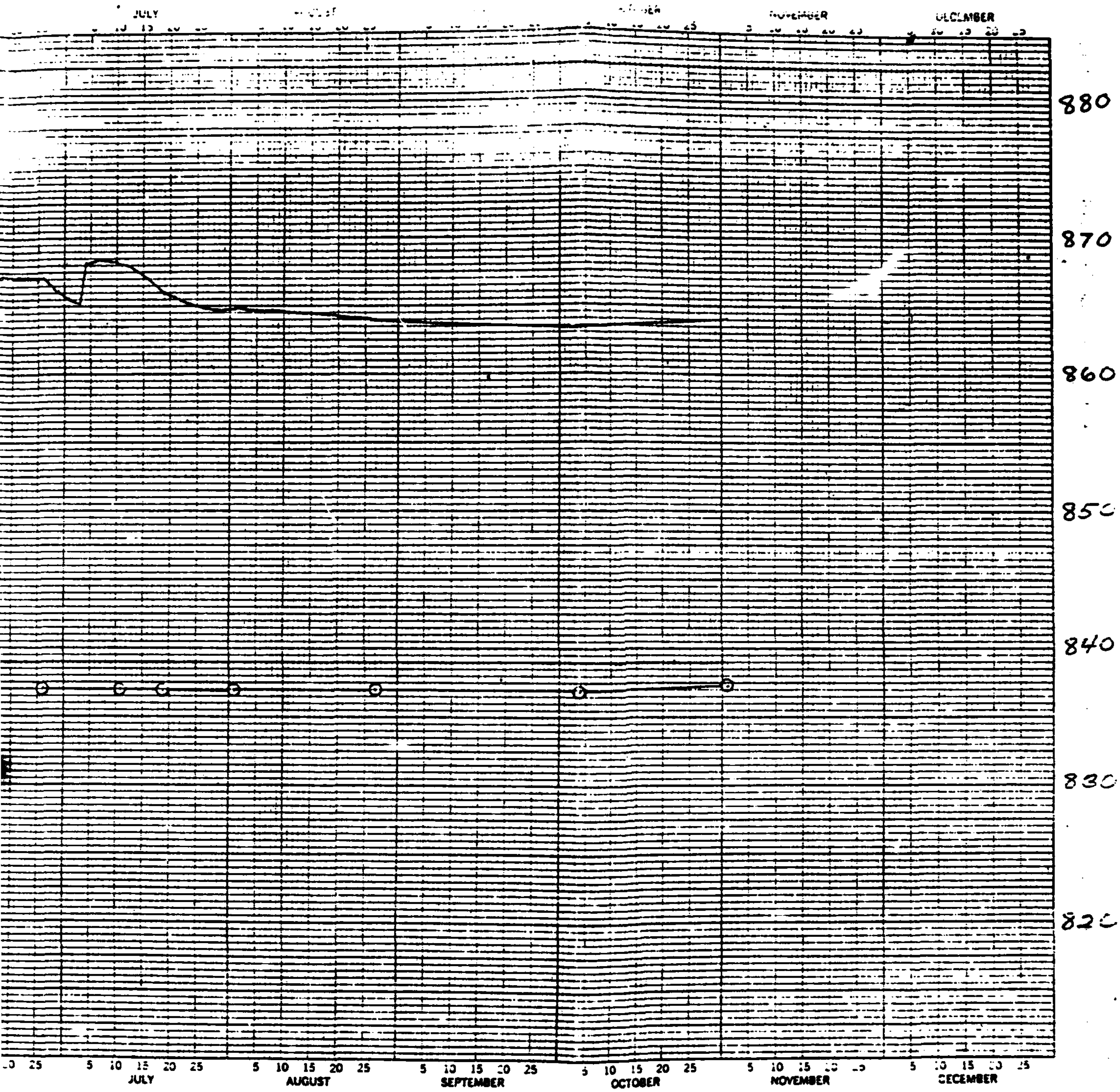
APPENDIX B Plate No. 14 RP-3-1734



47 2813

K+E 1 YEAR BY DAYS X 120 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

15-1-7



1984

D-519A

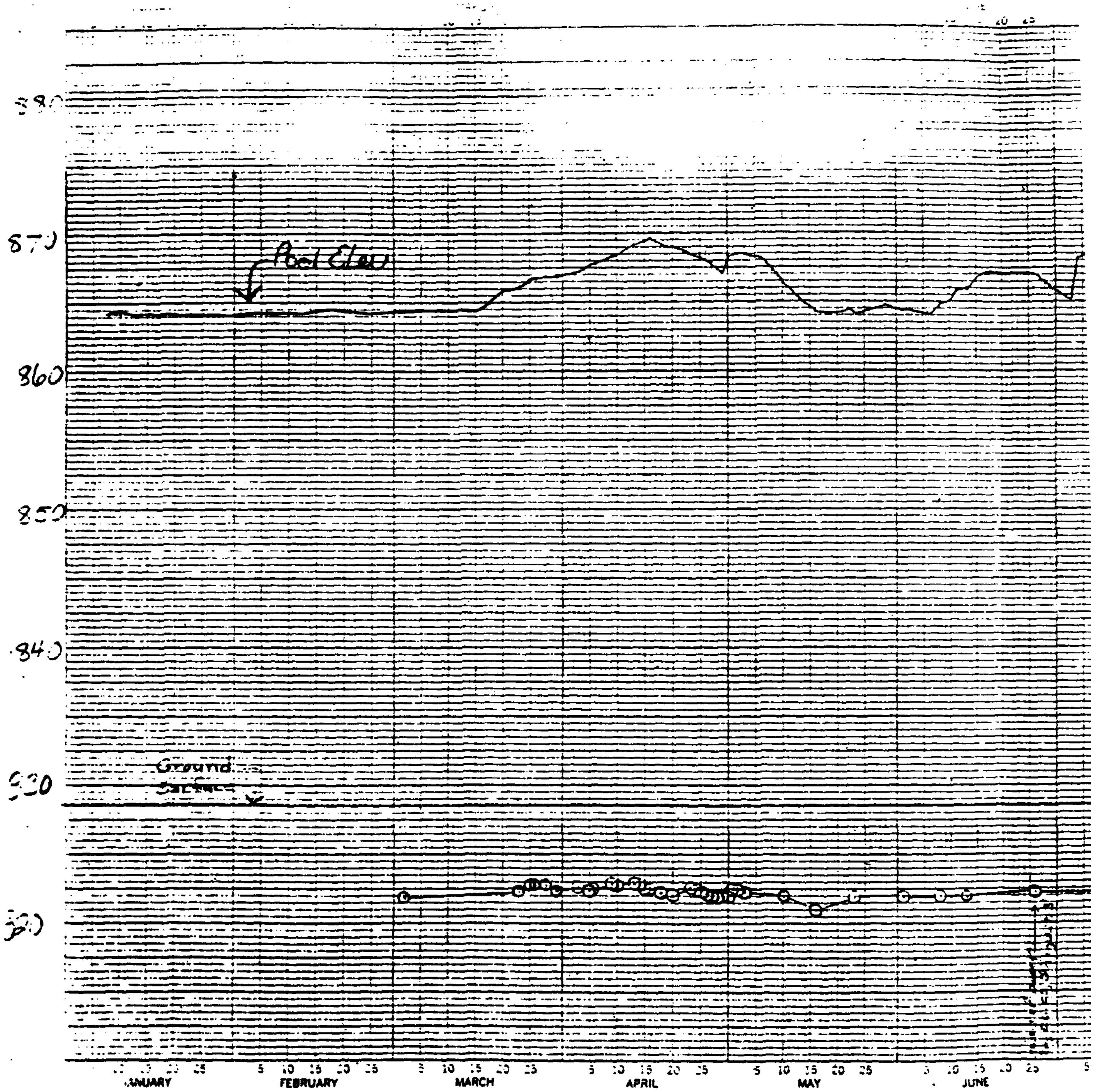
STA TIP EL 880.9

RNG GRND EL 882.9

MAT'L SDY LEAN CL RP-3-1735

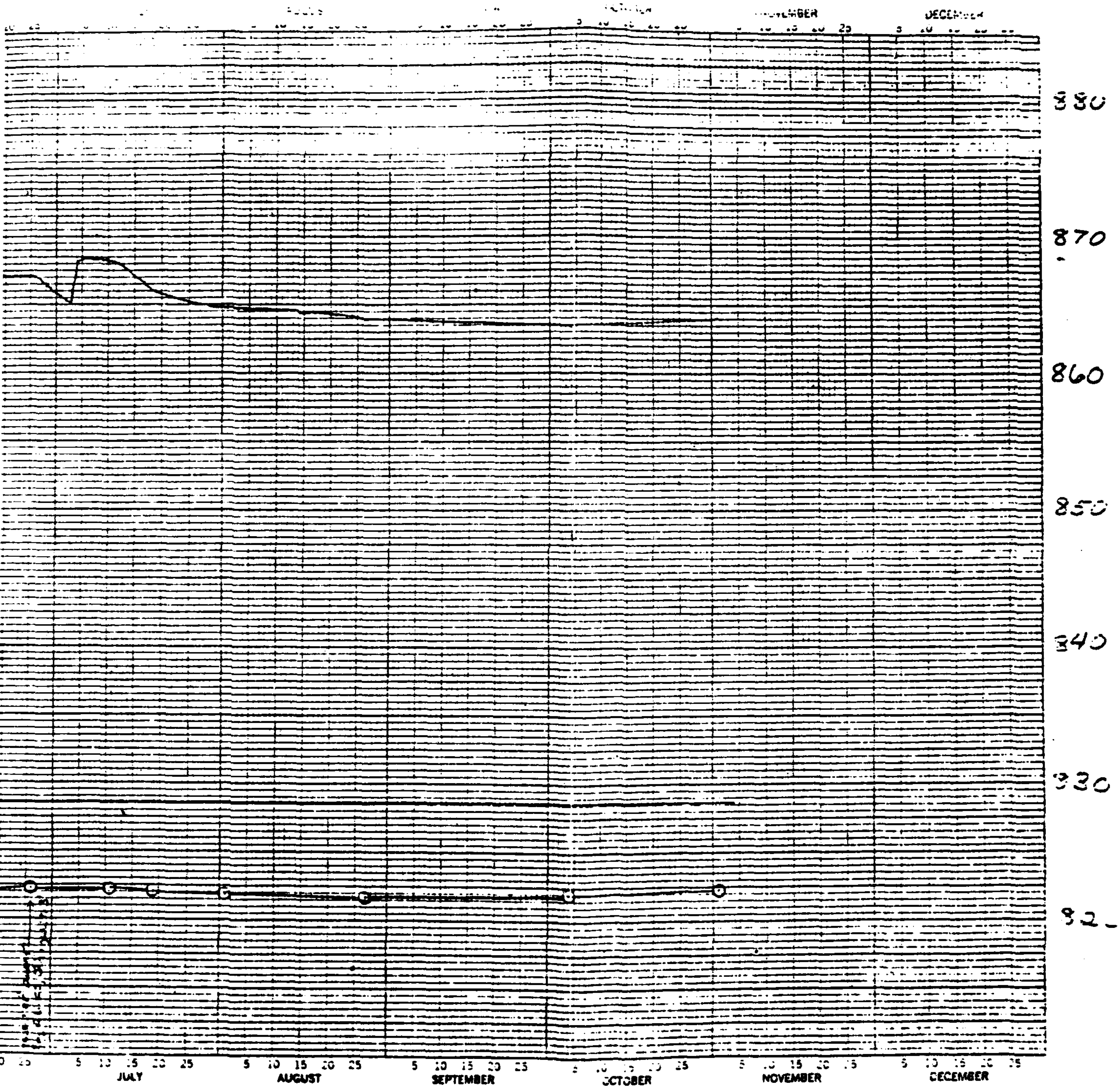
APPENDIX B Plate No. 15

47 2813



1984

15-1-7



1984

A-520

RP-3-1736

STA

TIP EL 812.1

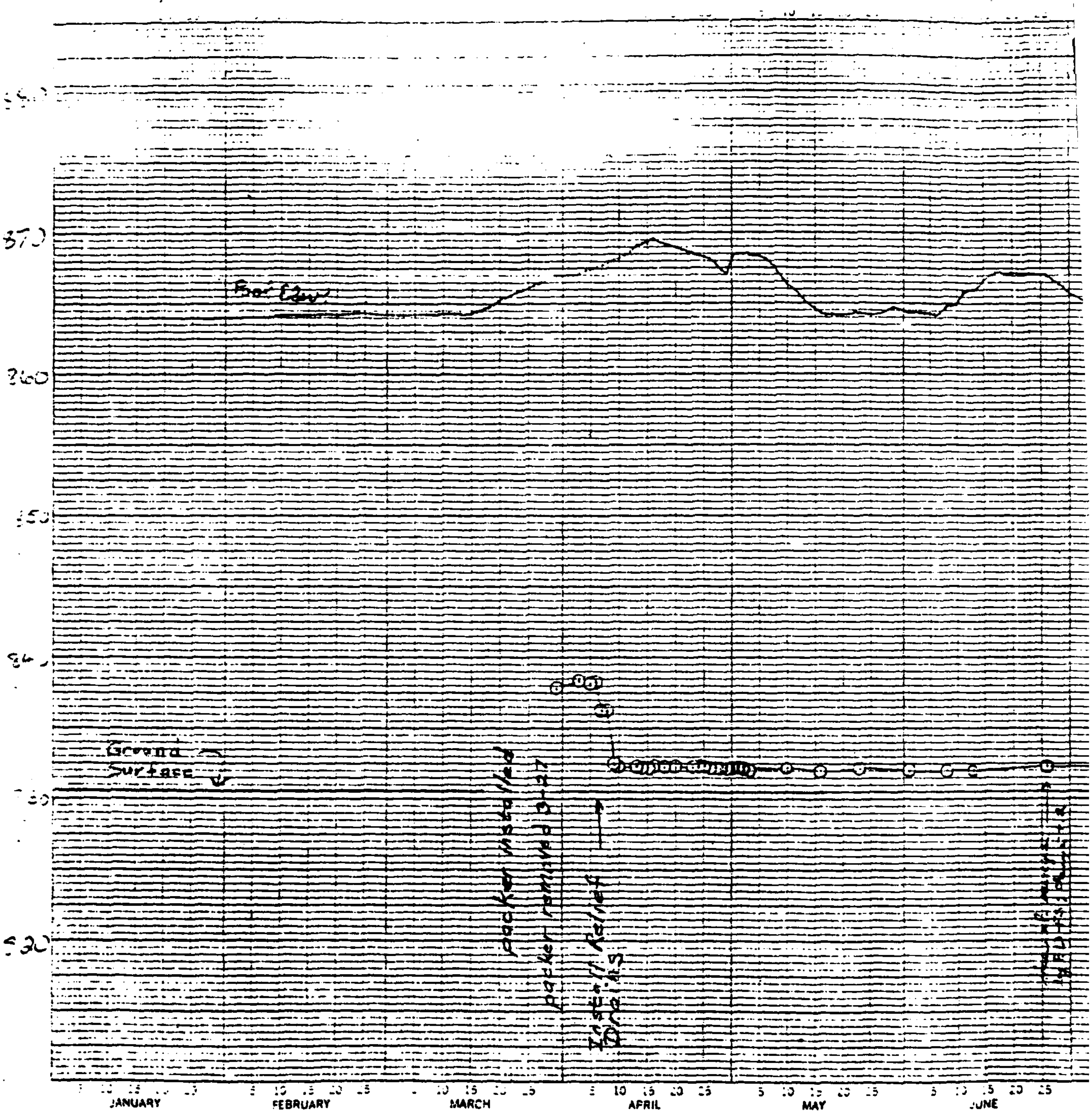
RNG

GRND EL 828.6

APPENDIX B PLATE NO. 16

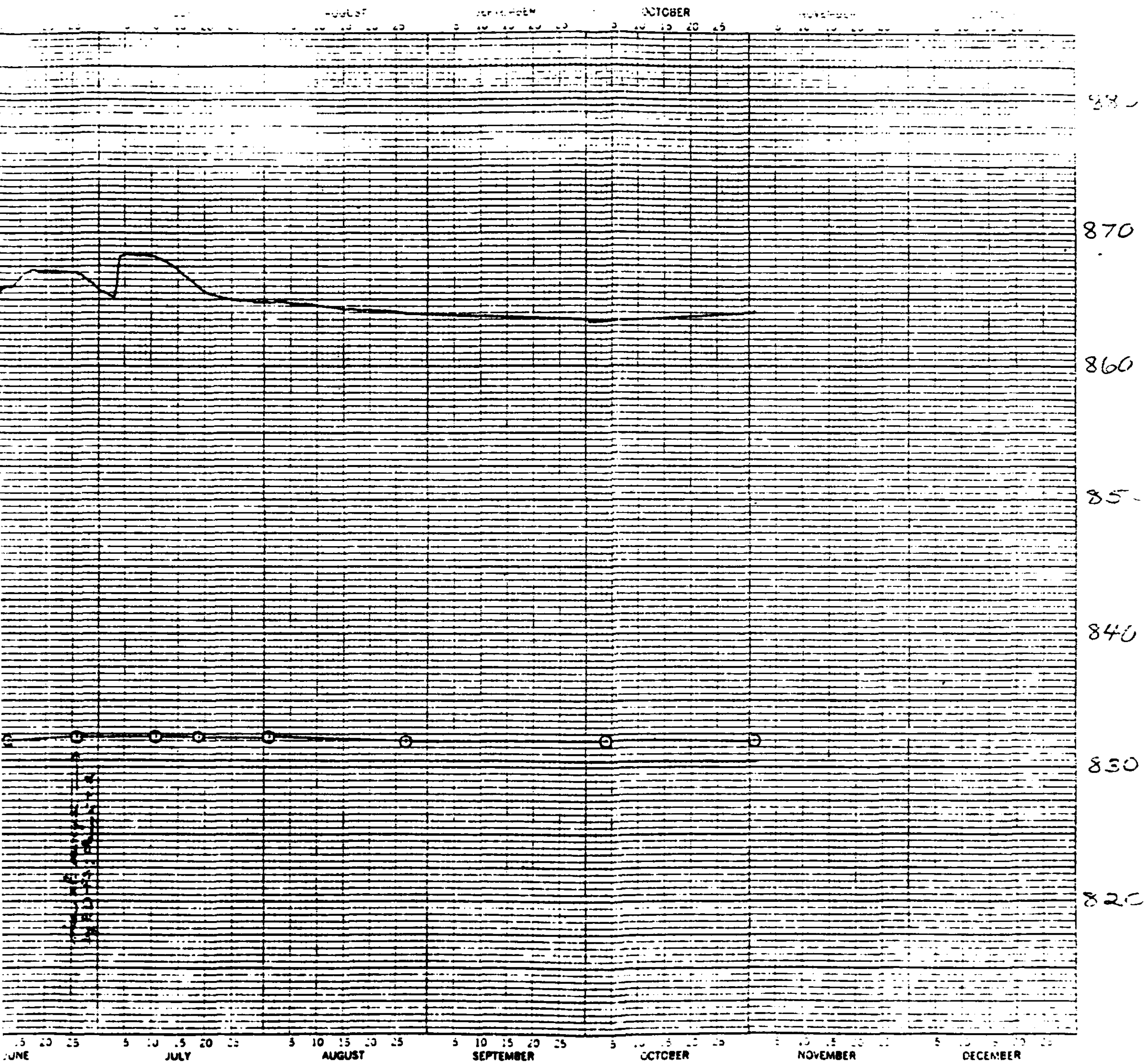
MAT'L SDY CLAY

47 2813



1984

15-1-7



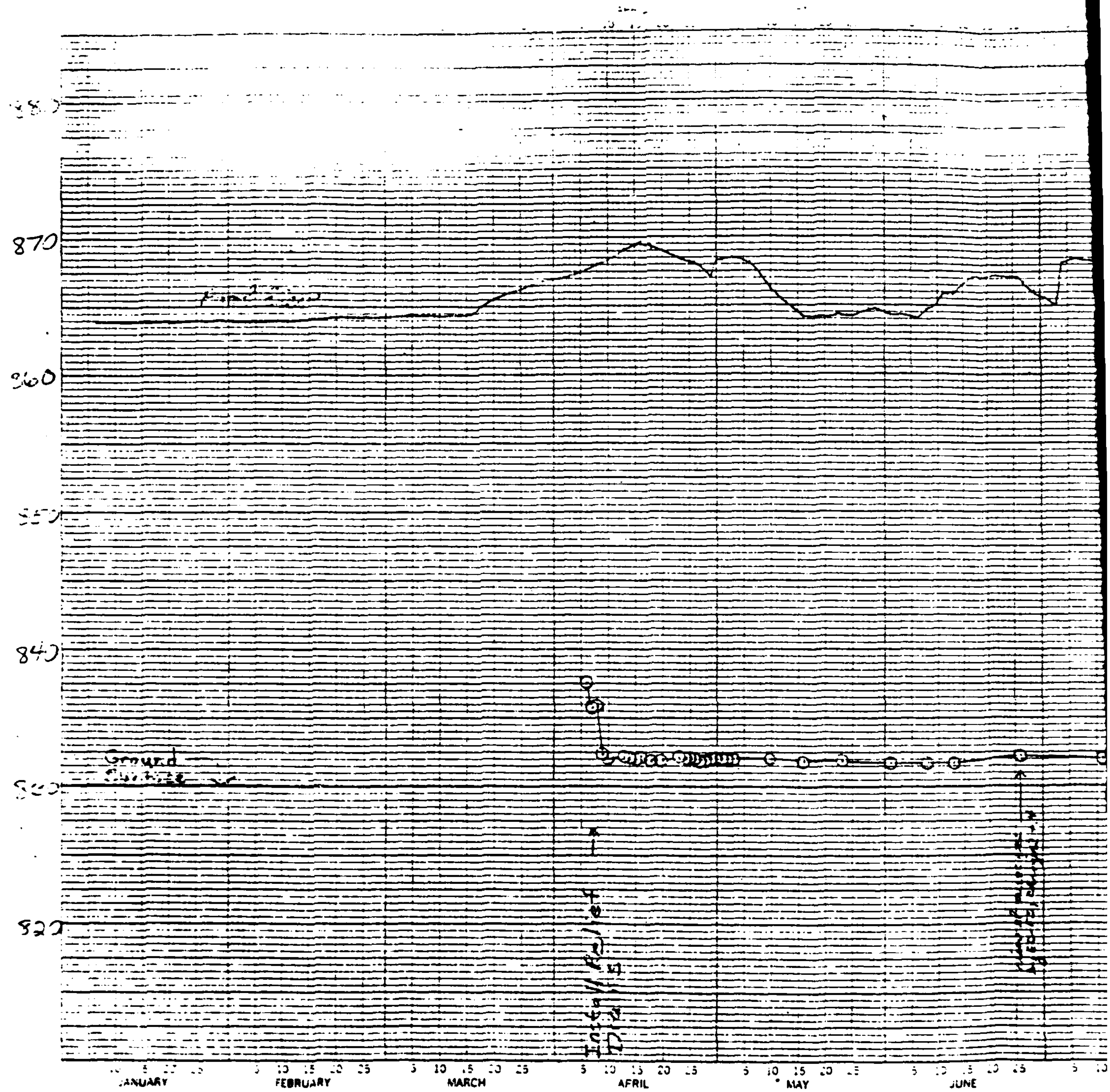
1984

A-521 RP-3-1737

STA TIP EL 810.2

RNG GRND EL 830.2

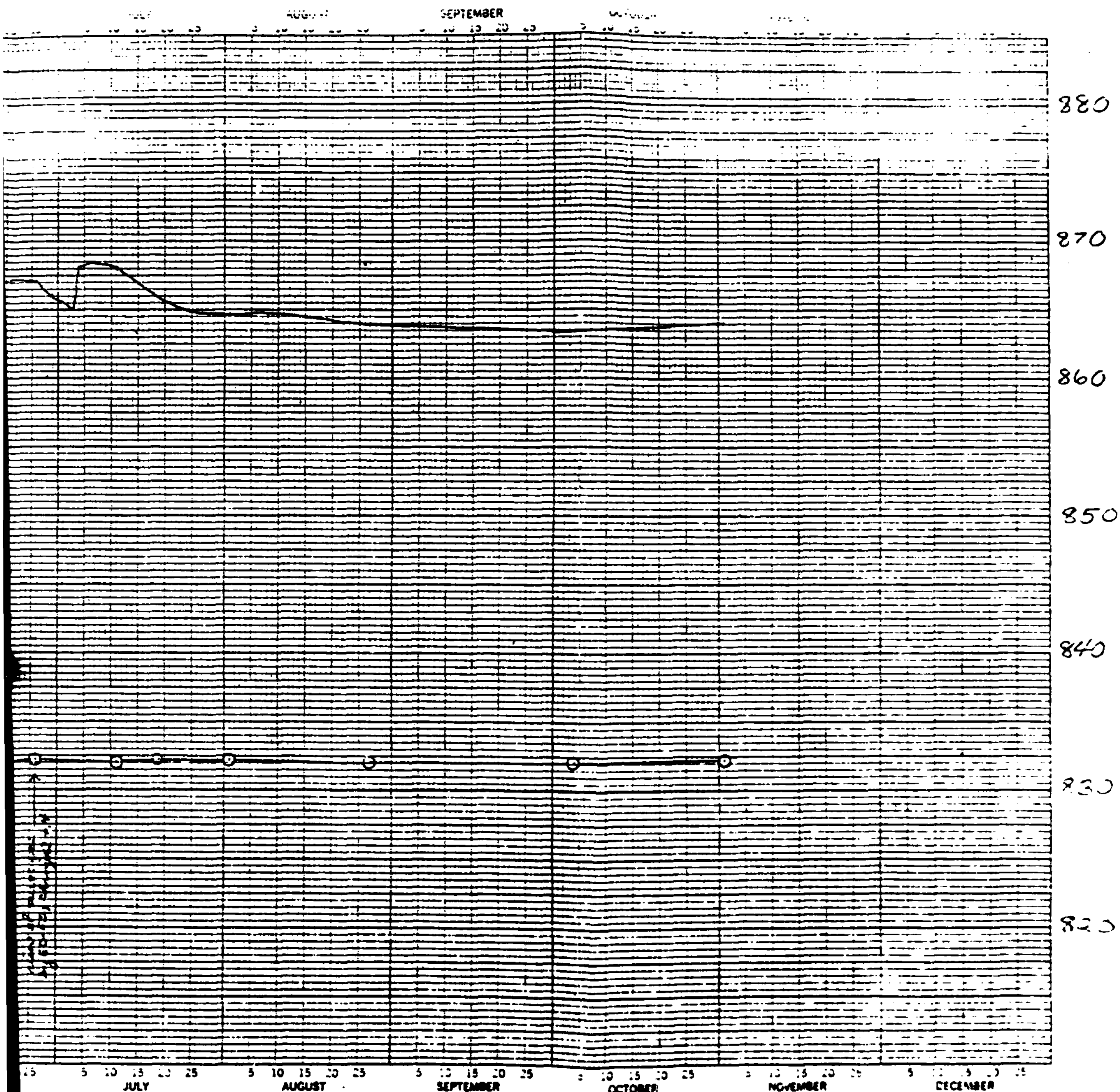
APPENDIX B PLATE NO. 17 MAT'L STY SAND


870
860
840
820
Ground
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE

1984

15-1-7

1



984

D-[illegible]

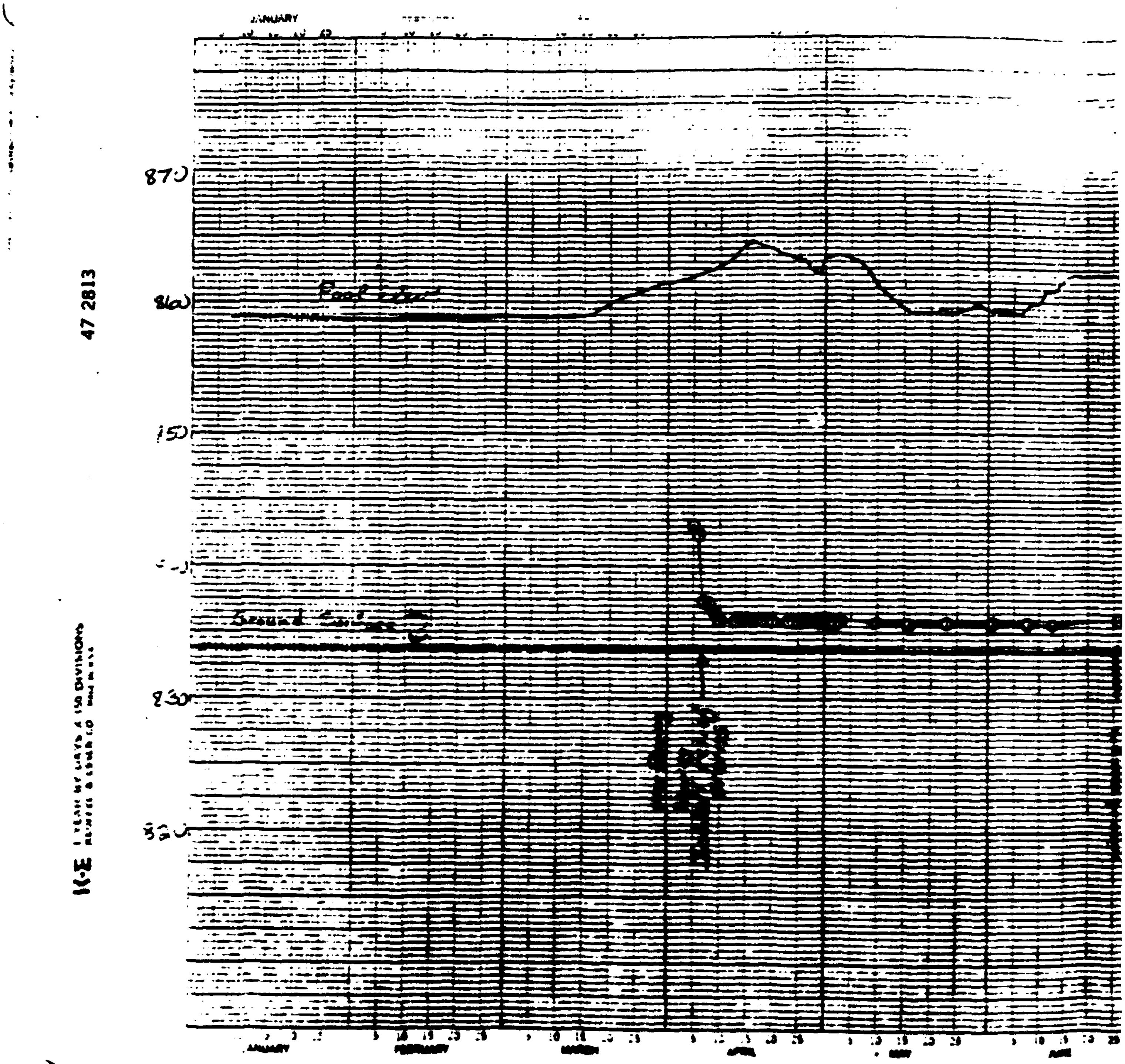
STA TIP EL 80[illegible]

RNG GRND EL 83[illegible]

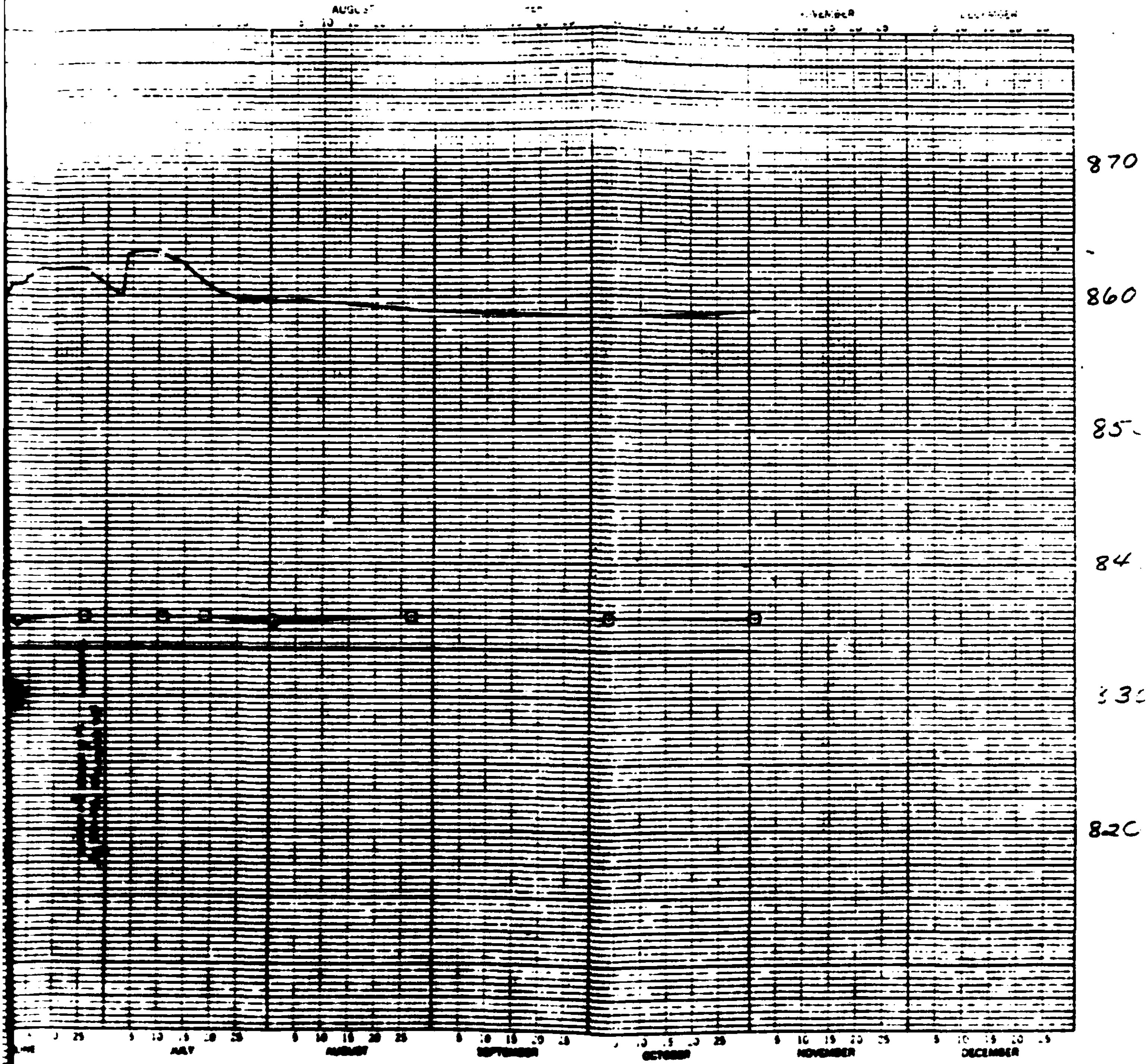
MAT'L [illegible] RP-3-1738

APPENDIX B Plate No. 18

9


JANUARY
870
860
850
830
820
Ground Surface
FEBRUARY
MARCH
APRIL
MAY
JUNE

15-1-7

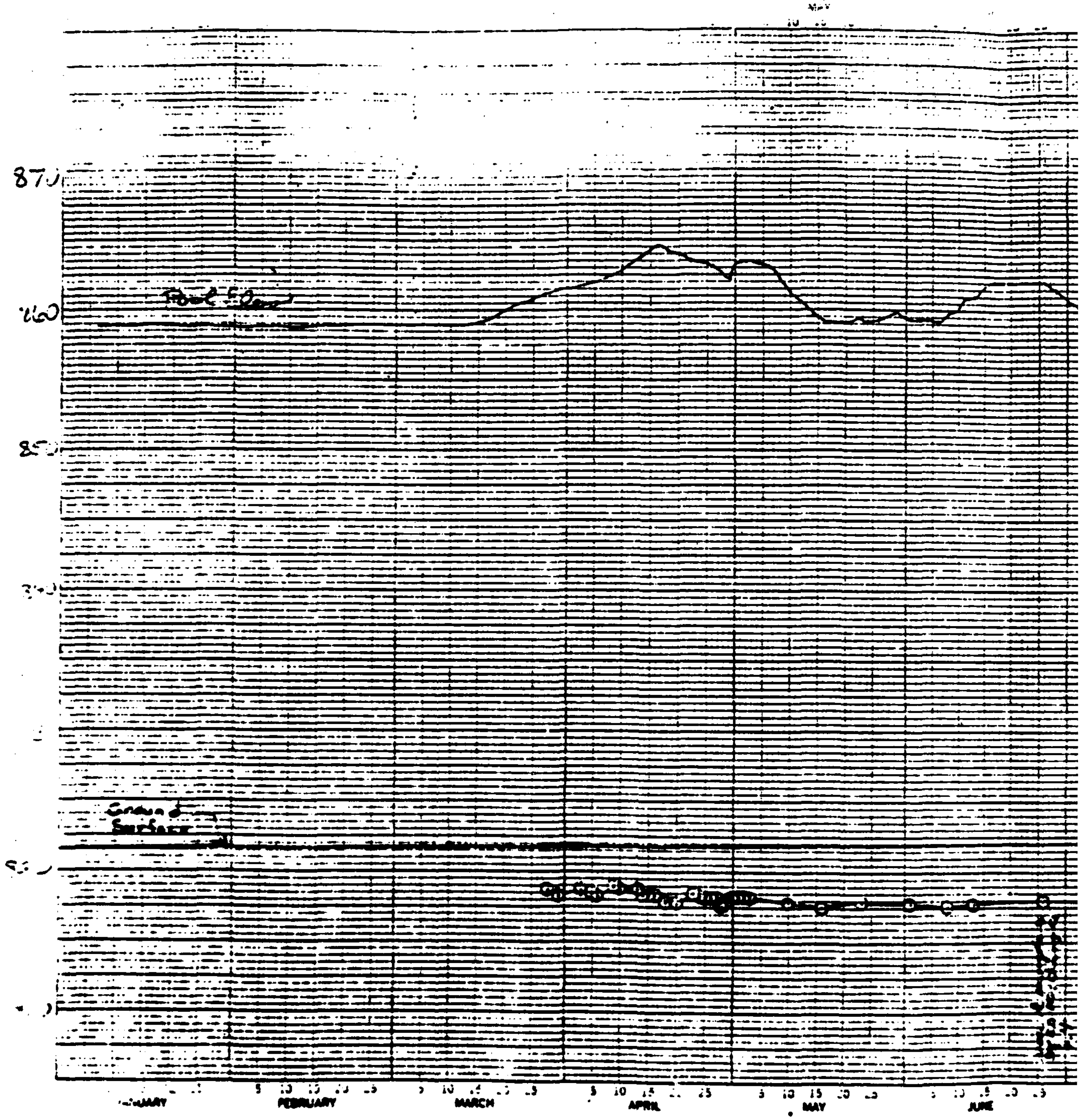


1984

STA
RNG
MAT'L CLY SAND

TIP EL 812.2
GRND EL 834.
RP-3-1739

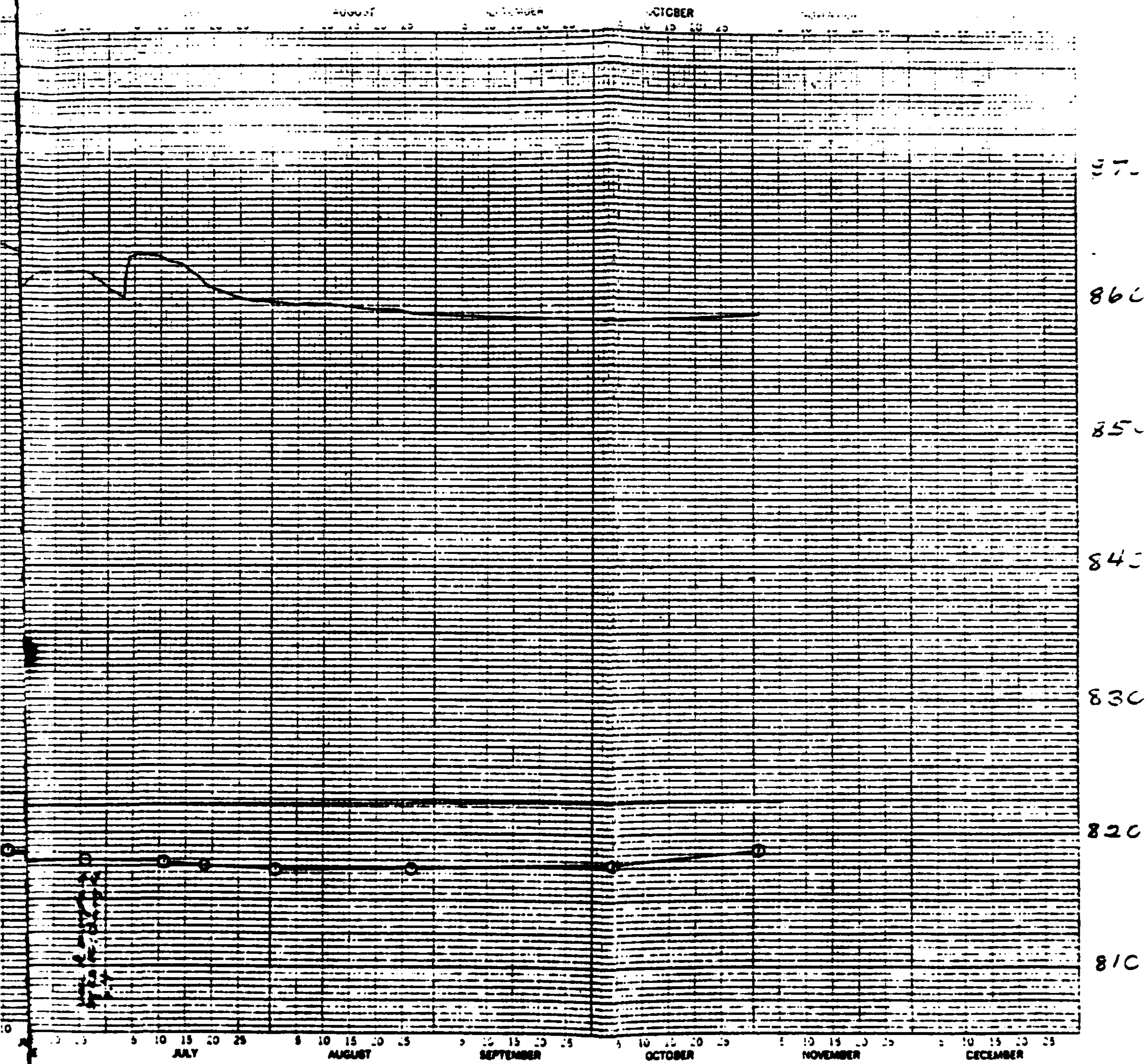
APPENDIX B Plate No. 19



870
860
850
Pool Elev
Ground Surface
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE

1984

10-1-7



1984

[illegible]

STA TIP EL [illegible]

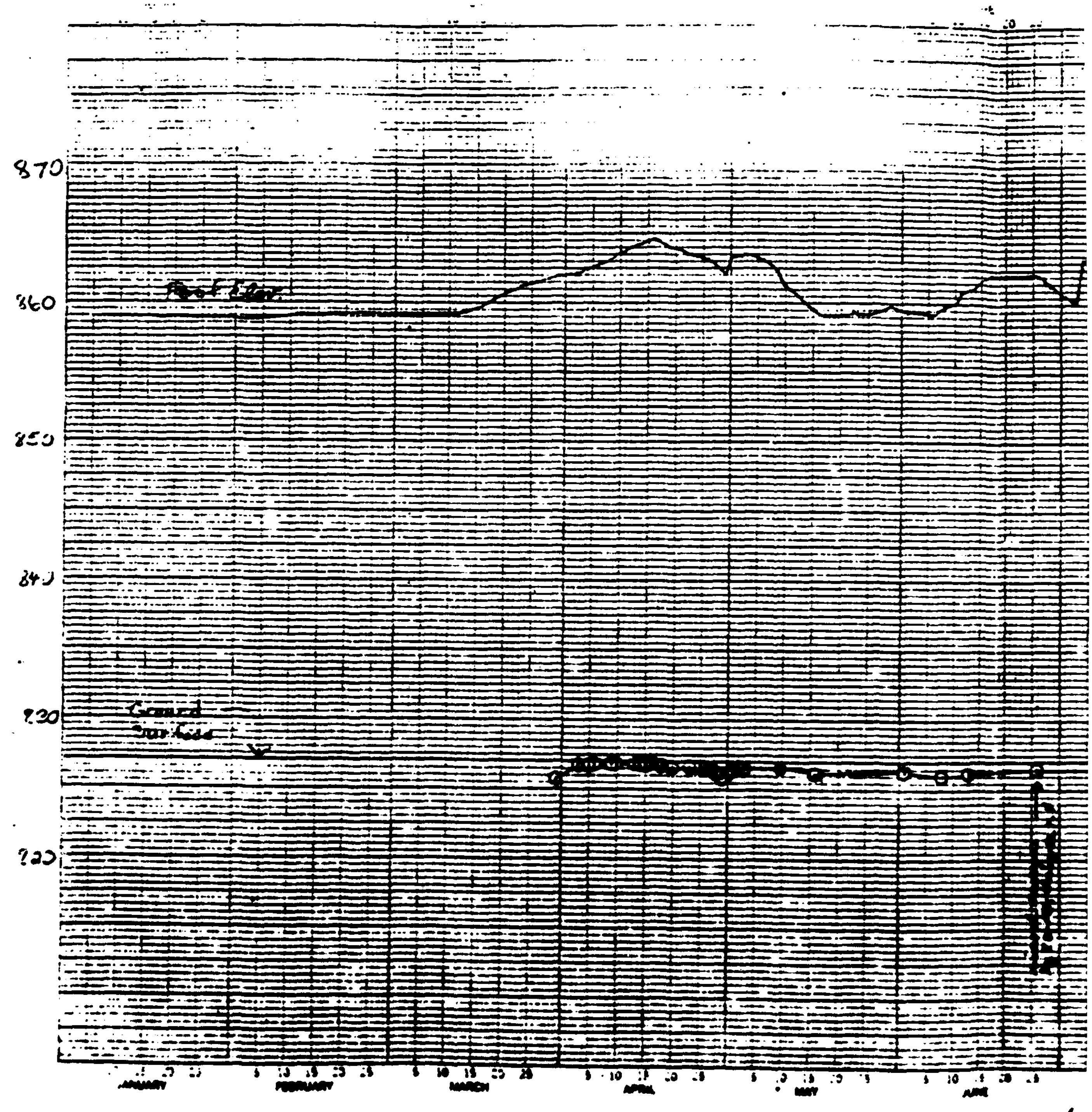
RNG GRND EL [illegible]

MAT'L SDY CLAY RP-3-1740

APPENDIX B Plate No. 20

9

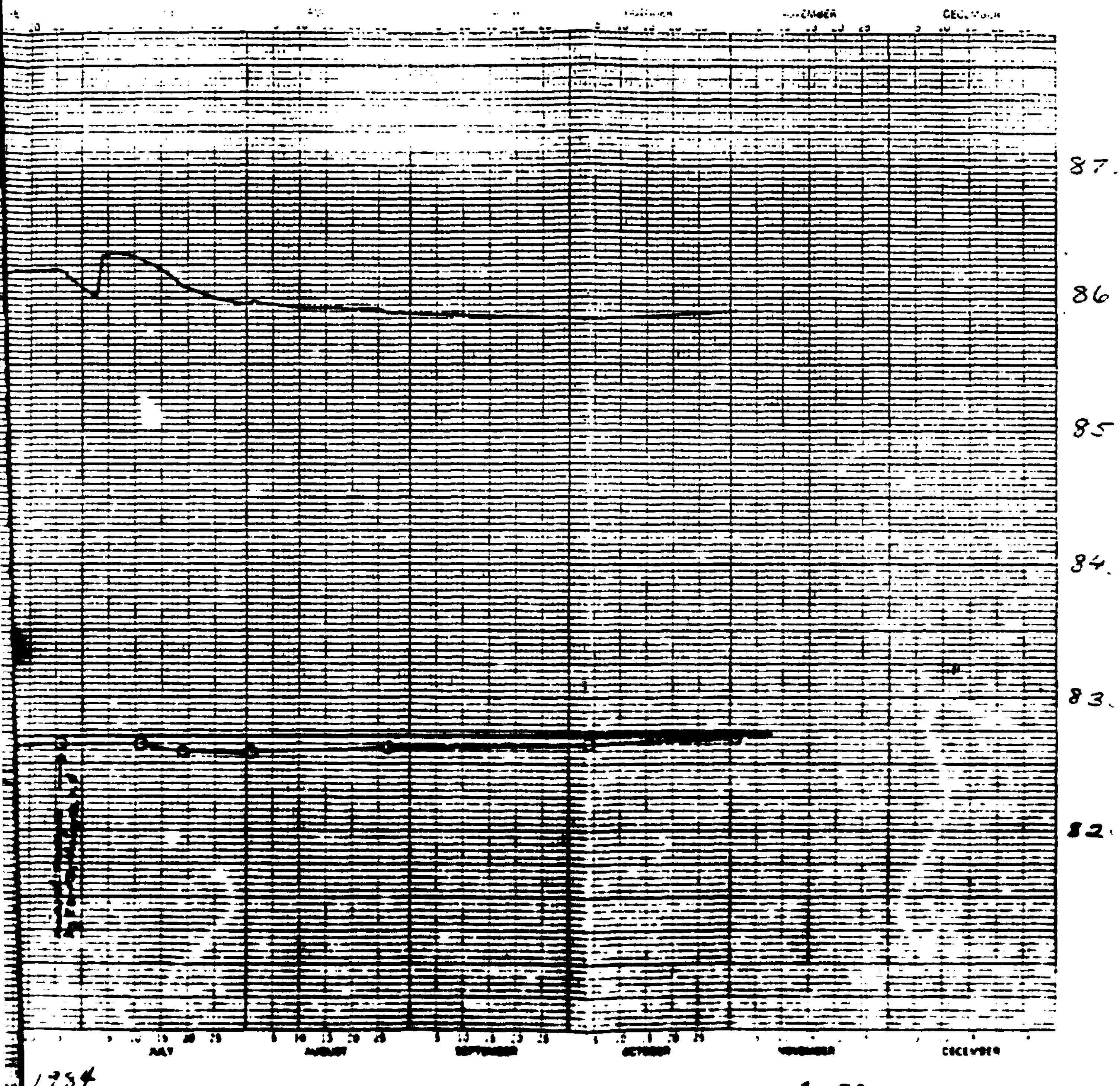
47 2813



K-E 1 YEAR BY DAYS X 110 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

1954

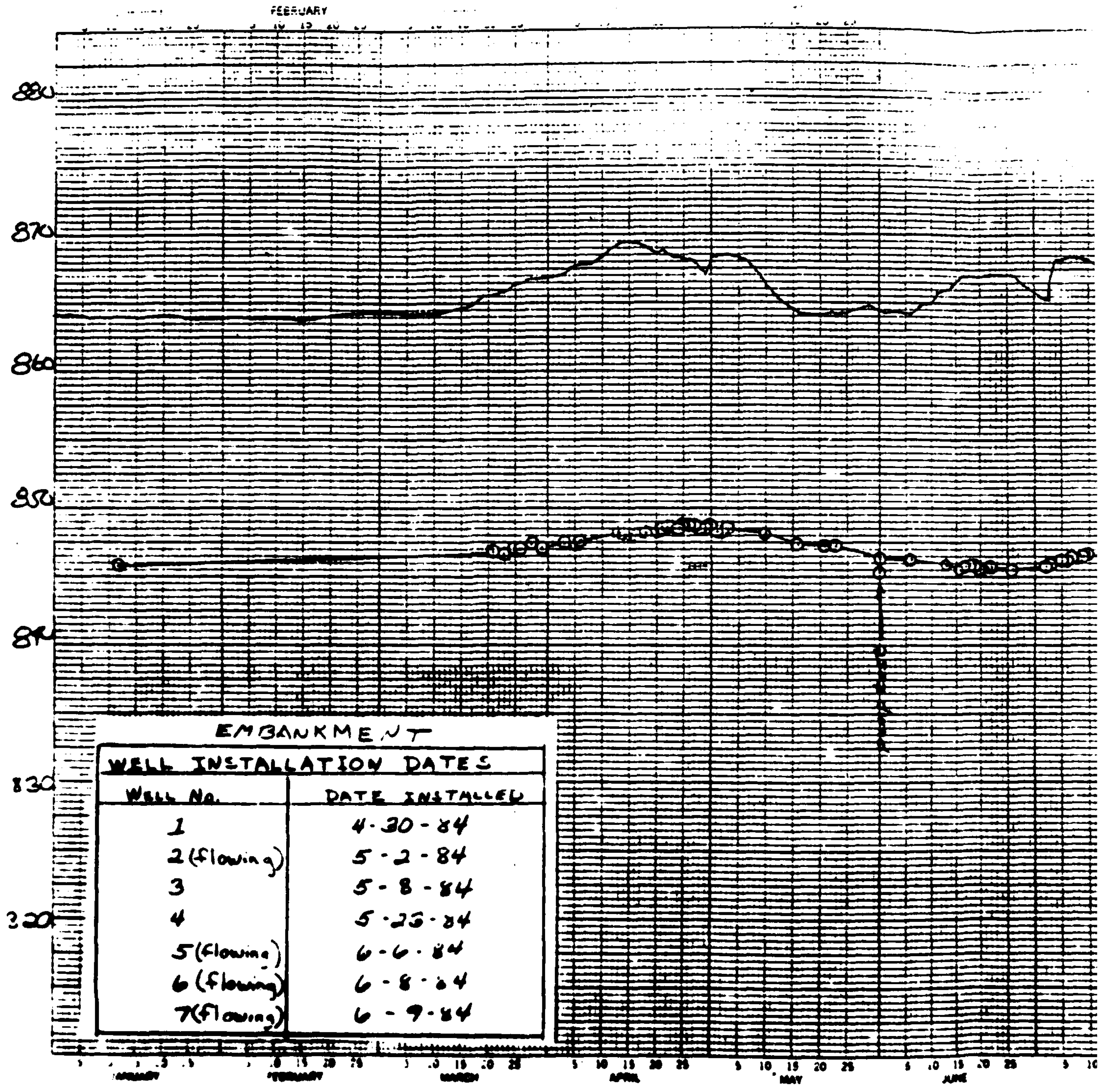
15-1-7



STA TIP EL
RING
MAT'L RP-3-1741
APPENDIX B Plate No 21

1

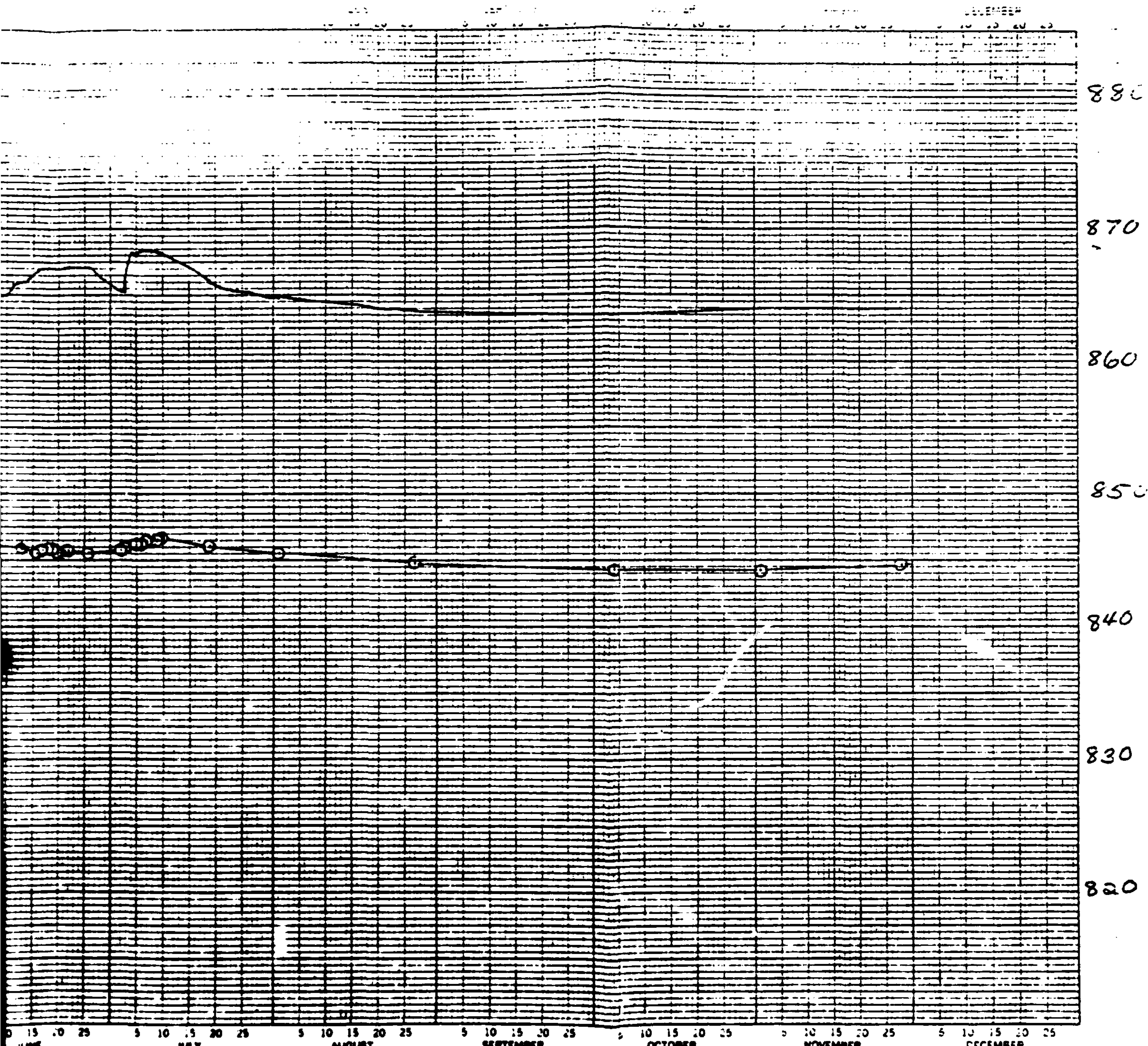
2



EMBANKMENT

WELL INSTALLATION DATES	
WELL No.	DATE INSTALLED
1	4-30-84
2 (flowing)	5-2-84
3	5-8-84
4	5-23-84
5 (flowing)	6-6-84
6 (flowing)	6-8-84
7 (flowing)	6-9-84

15-1-7

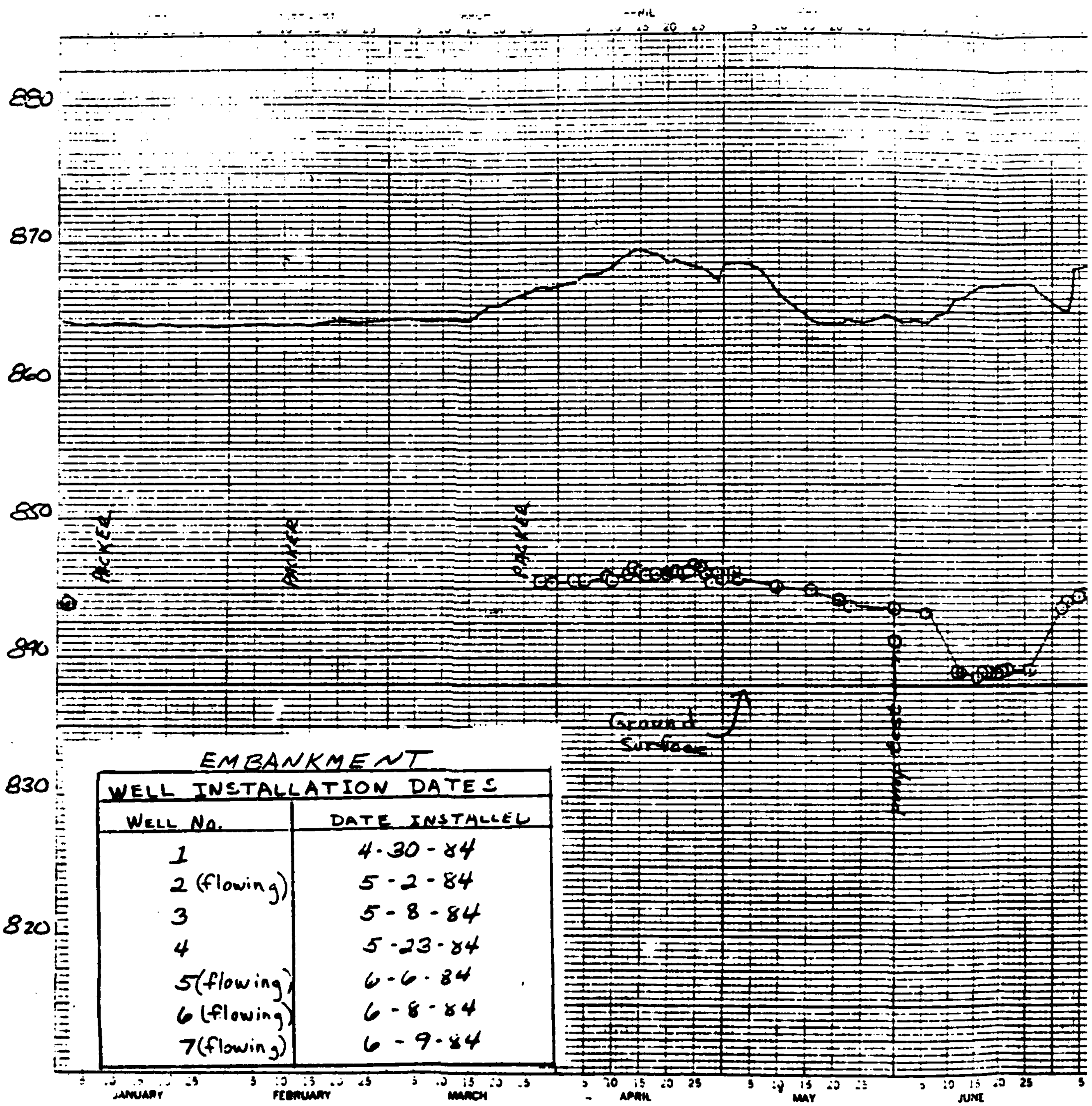


1984

P-106-4

STA 106+02 TIP EL 807.9
RNG 15D GRND EL 894.9
MAT'L SIY GRV SD RP-3-1742

APPENDIX B Plate No. 22



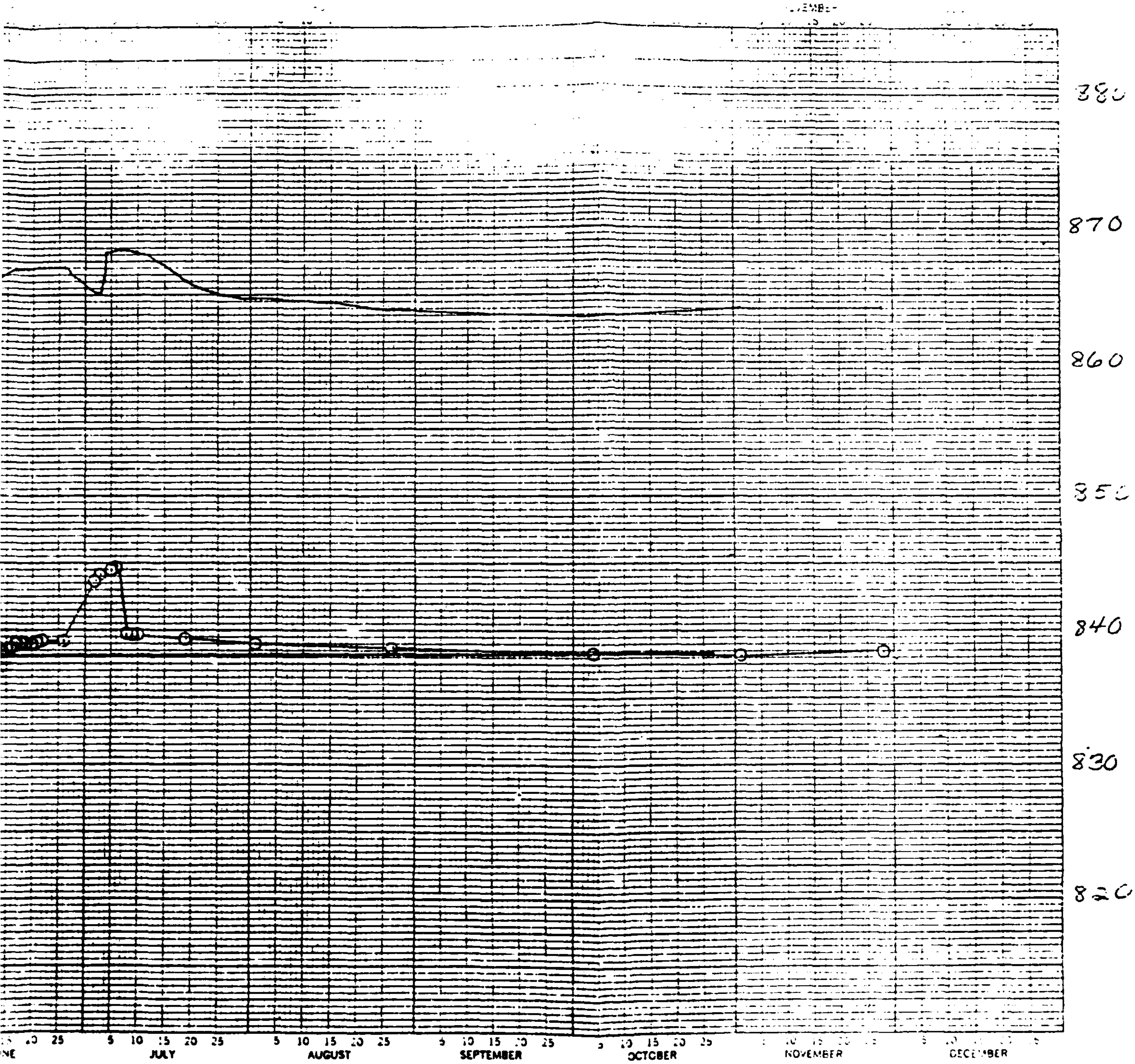
EMBANKMENT

WELL INSTALLATION DATES

WELL No.	DATE INSTALLED
1	4-30-84
2 (flowing)	5-2-84
3	5-8-84
4	5-23-84
5 (flowing)	6-6-84
6 (flowing)	6-8-84
7 (flowing)	6-9-84

1984

15-1-7



1984

P-112-6

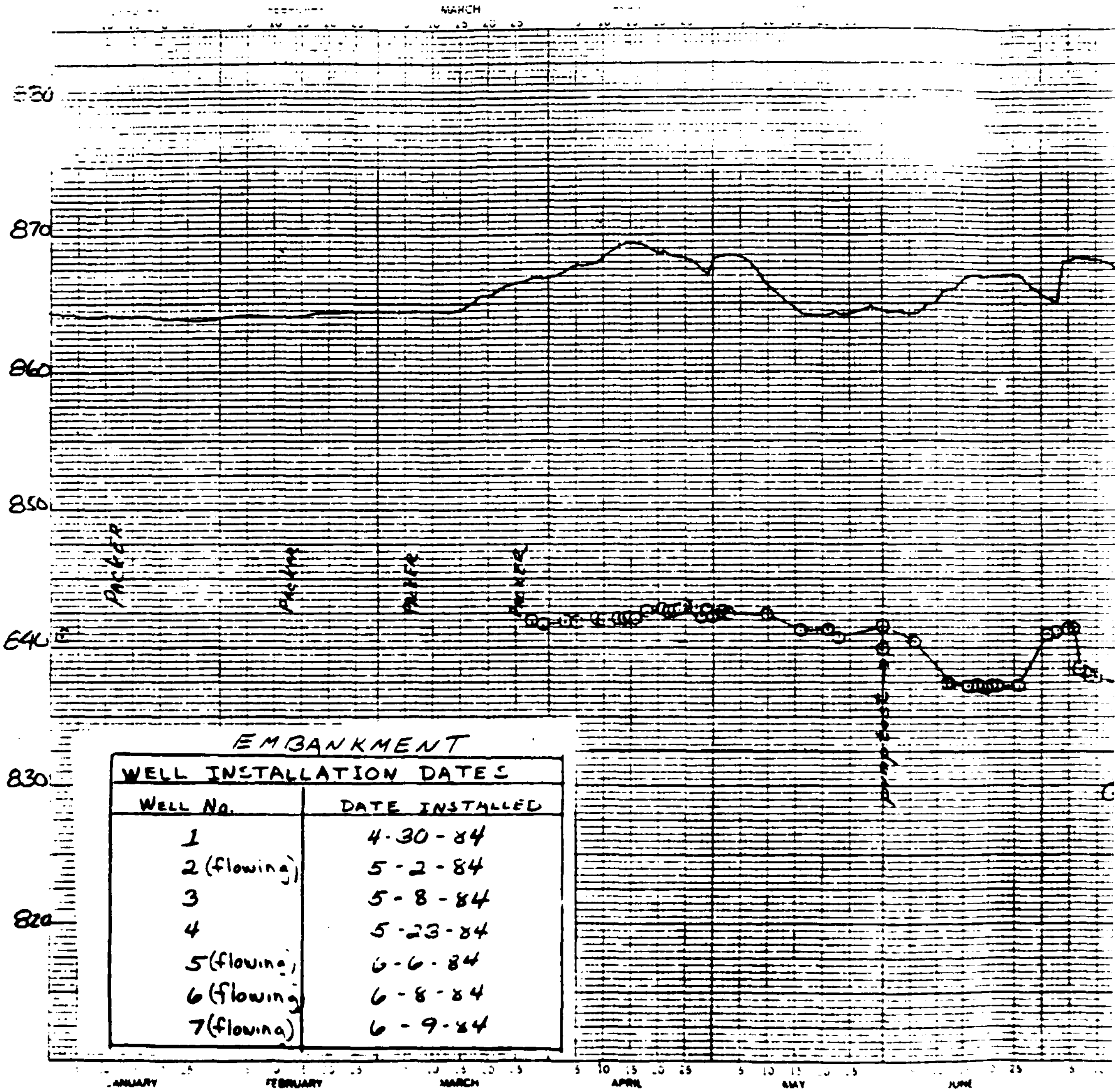
STA 110+00 TIP EL 808.1

RNG 348 D GRND EL 868.1

MAT'L SDY CL + GRV RP3-1743

APPENDIX B PLATE NO. 23

2



EMBANKMENT

WELL INSTALLATION DATES	
WELL No.	DATE INSTALLED
1	4-30-84
2 (flowing)	5-2-84
3	5-8-84
4	5-23-84
5 (flowing)	6-6-84
6 (flowing)	6-8-84
7 (flowing)	6-9-84

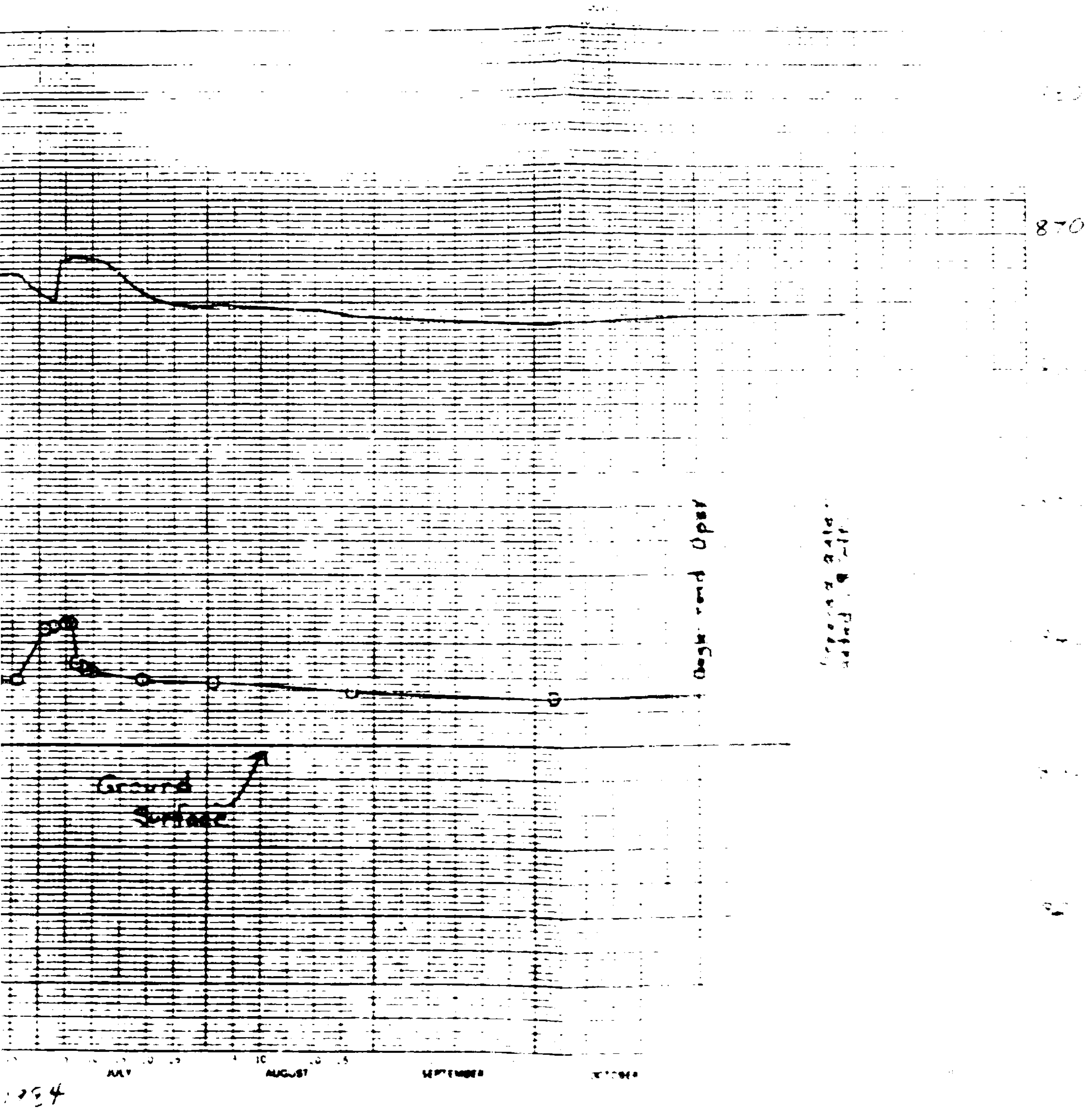
1984

15-1-7

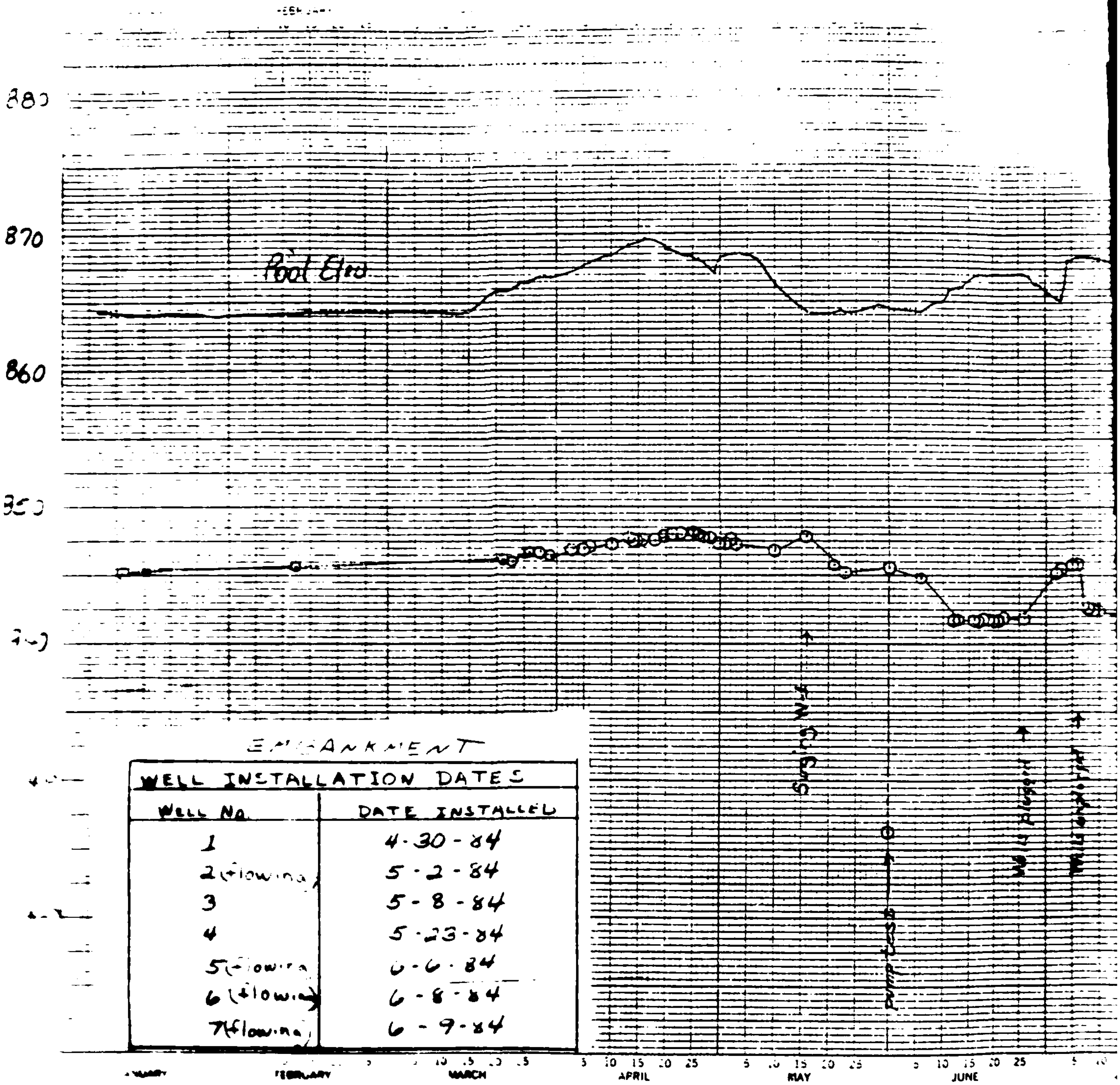
47 2813

K+E 1 YEAR BY DAYS X 100 DIVISIONS

1



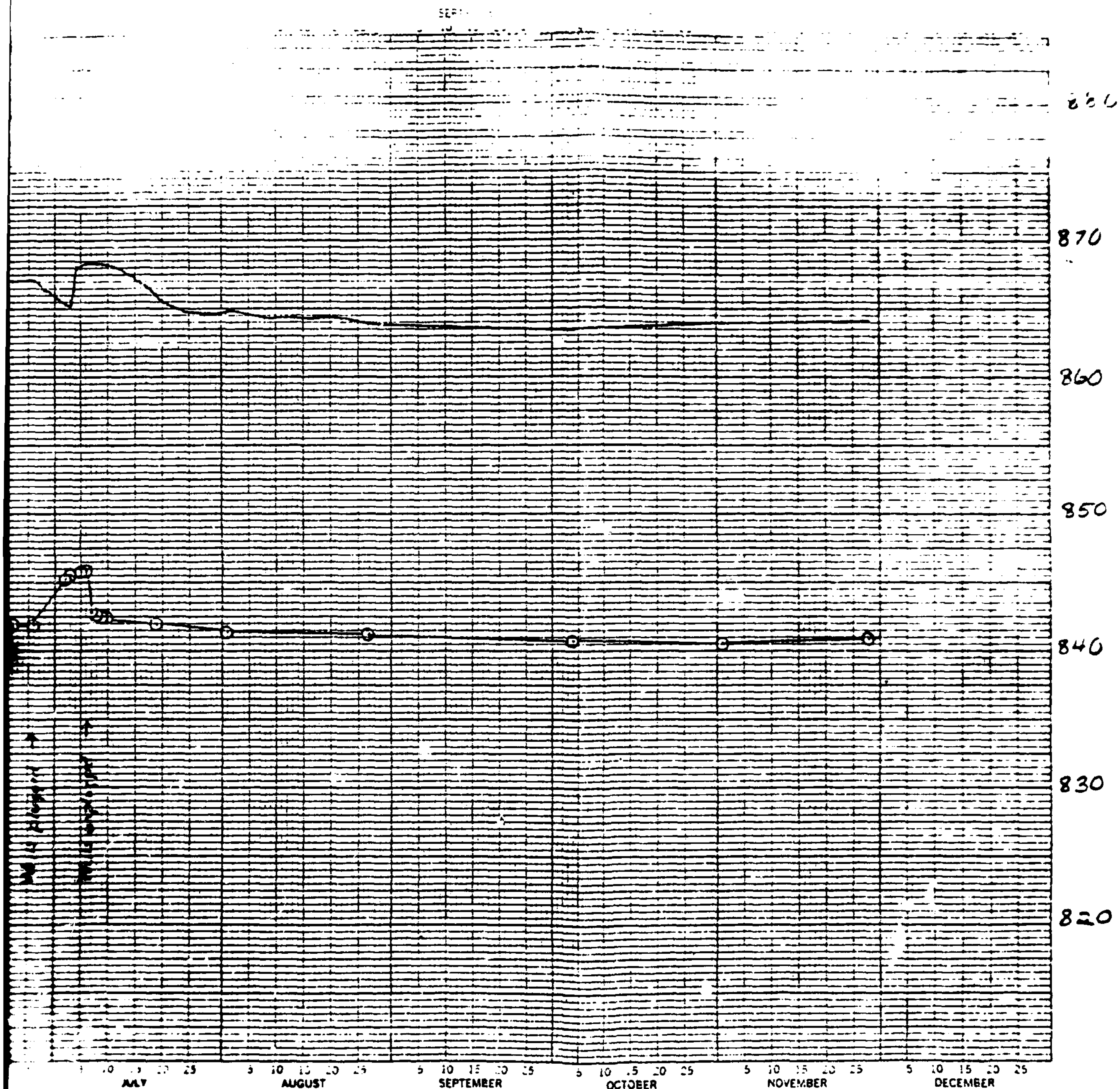
870
Ground Surface
JULY
AUGUST
SEPTEMBER



WELL INSTALLATION DATES

WELL No.	DATE INSTALLED
1	4-30-84
2 (flowing)	5-2-84
3	5-8-84
4	5-23-84
5 (flowing)	6-6-84
6 (flowing)	6-8-84
7 (flowing)	6-9-84

15-1-9

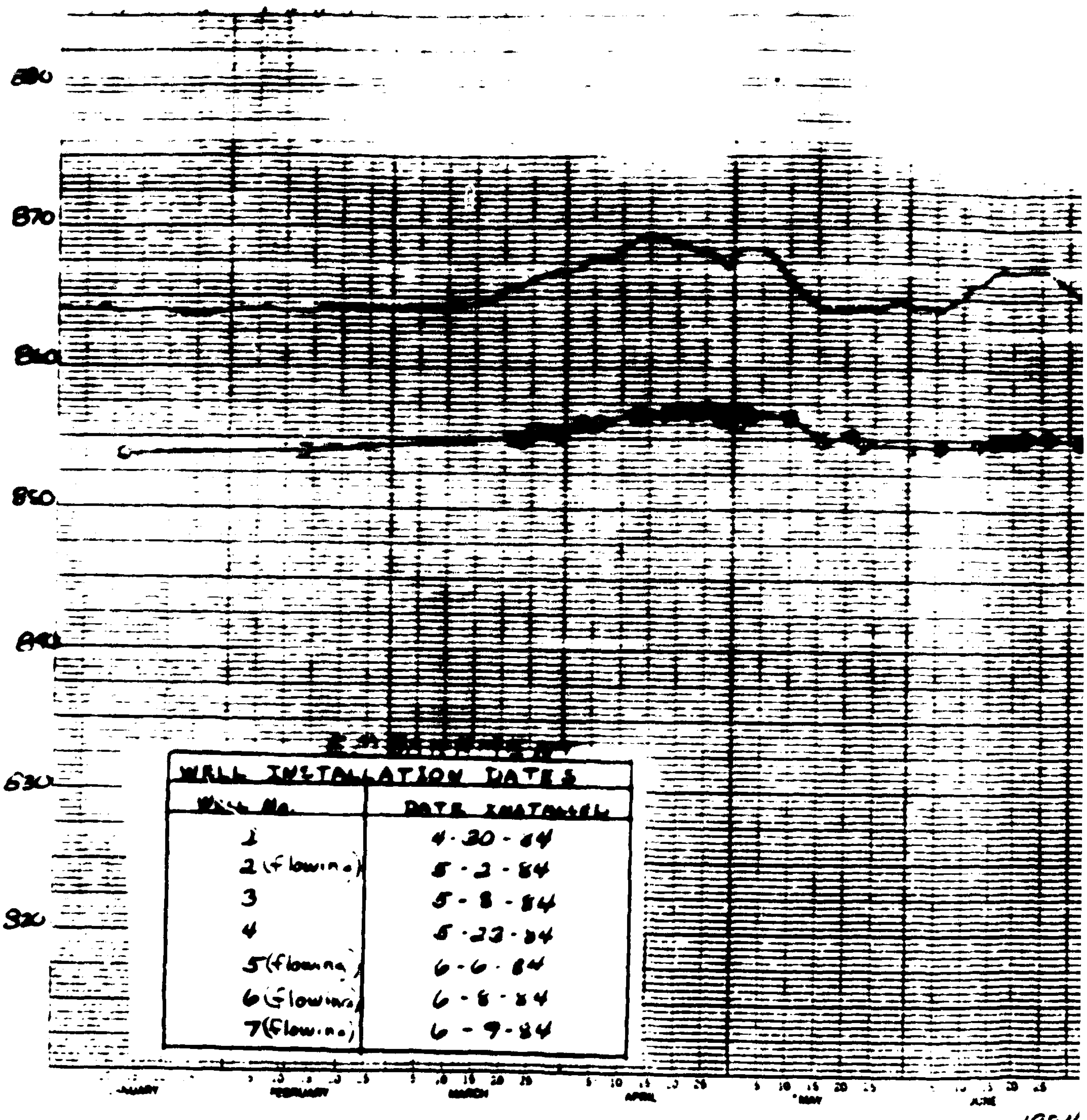


P-110-8

STA 110+00 TIP EL 811.0

RP-3-1745 RNG 109 D GRND EL 863.0

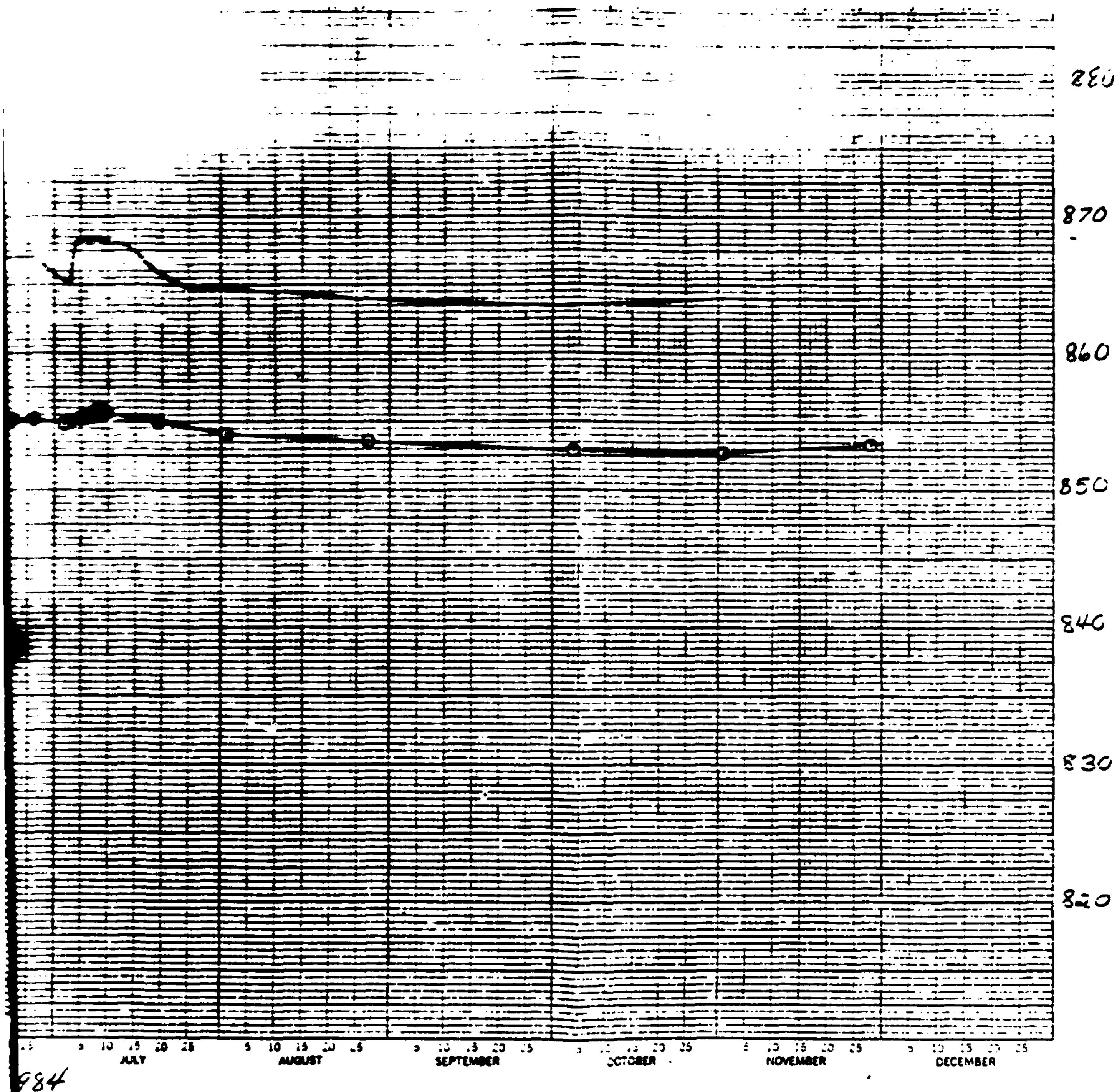
APPENDIX B PLATE NO. 25 MAT'L CLY SDY GRV



WELL INSTALLATION DATES

Well No.	DATE INSTALLED
1	4-30-84
2 (flowing)	5-2-84
3	5-8-84
4	5-22-84
5 (flowing)	6-6-84
6 (flowing)	6-8-84
7 (flowing)	6-9-84

15-1-7



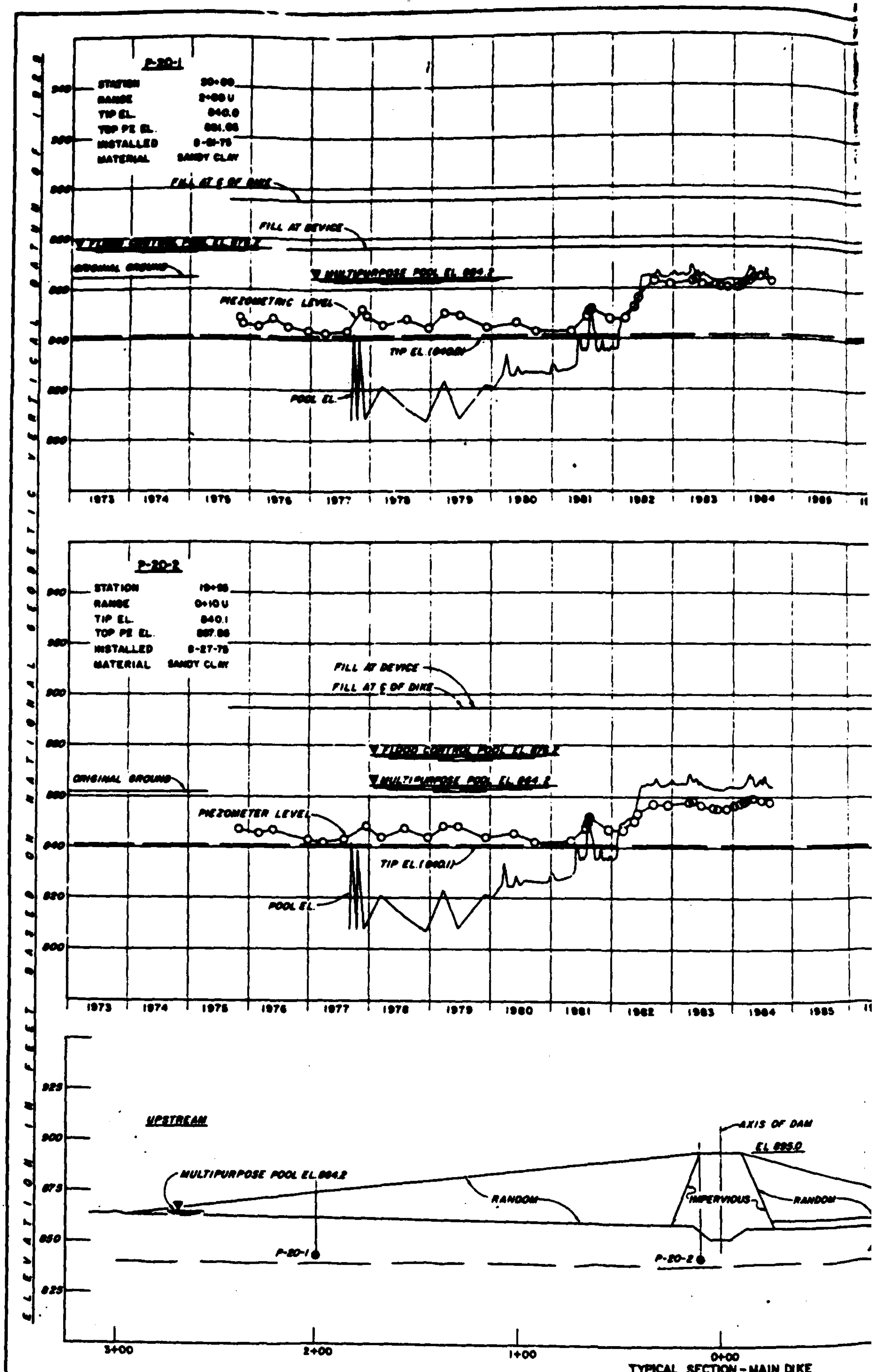
P-118-1

STA 118+00 TIP EL 307.0

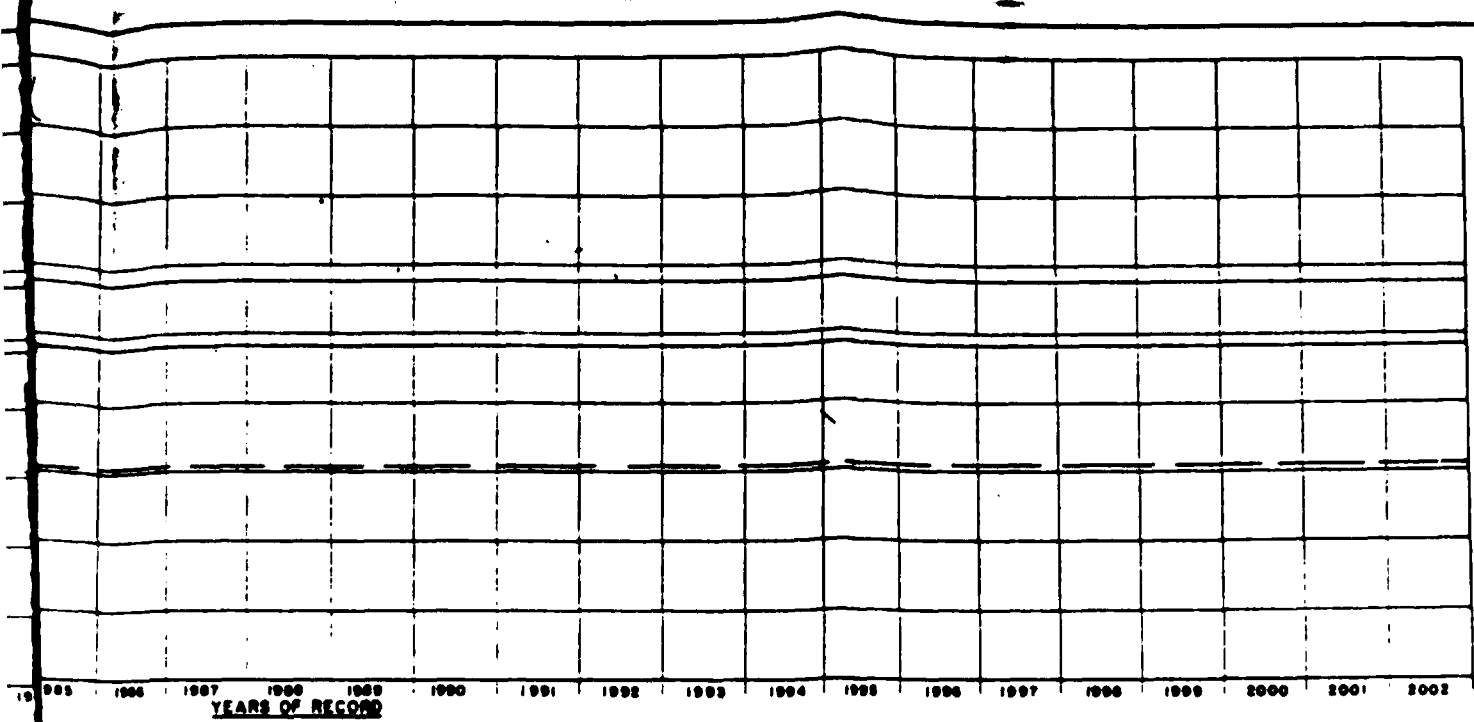
RNG 100'D GRND EL 868.5

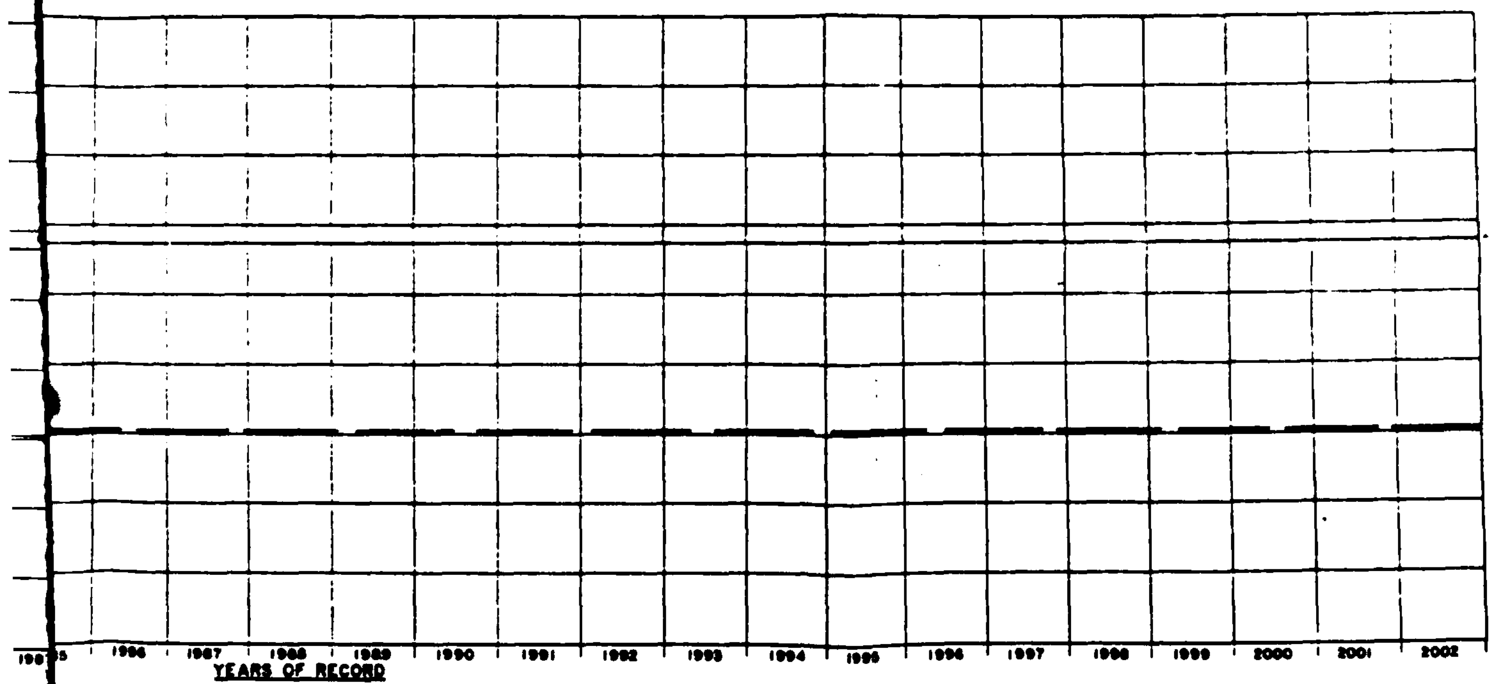
MAT'L GRVY CLY SD

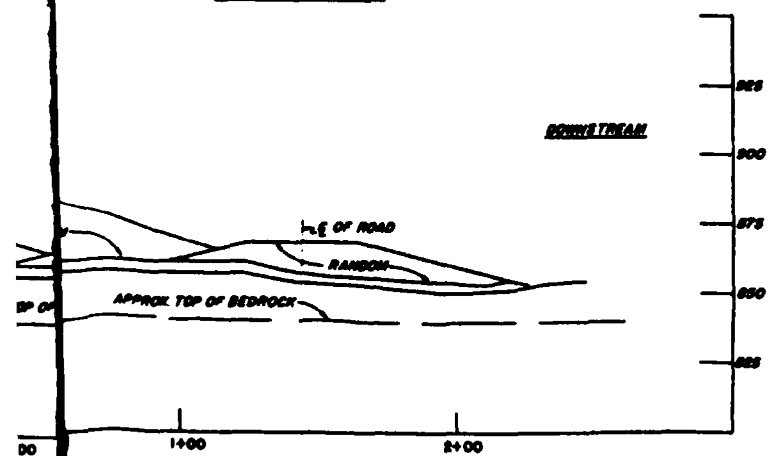
RP-3-1746 2 Plate No. 26


ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929
P-20-1
STATION
RANGE
TIP EL.
TOP PE EL.
INSTALLED
MATERIAL
SANDY CLAY
FILL AT ℄ OF DIKE
FILL AT DEVICE
ORIGINAL GROUND
MULTIPURPOSE POOL EL 864.2
PIEZOMETRIC LEVEL
TIP EL.
POOL EL.
1973
1974
1975
1976
1977
1978
1979
1980
1981
1982
1983
1984
1985
P-20-2
STATION
RANGE
TIP EL.
TOP PE EL.
INSTALLED
MATERIAL
19+88
0+10 U
840.1
SANDY CLAY
FILL AT DEVICE
FILL AT ℄ OF DIKE
FLOOD CONTROL POOL EL
MULTIPURPOSE POOL EL 864.2
ORIGINAL GROUND
PIEZOMETER LEVEL
TIP EL.
POOL EL.
1973
1974
1975
1976
1977
1978
1979
1980
1981
1982
1983
1984
1985
925
900
875
850
825
UPSTREAM
AXIS OF DAM
EL 895.0
MULTIPURPOSE POOL EL 864.2
RANDOM
IMPERVIOUS
RANDOM
P-20-1
P-20-2
3+00
2+00
1+00
0+00
TYPICAL SECTION - MAIN DIKE

15-1-7







REVISED SEPTEMBER 1984
LITTLE PLATTE RIVER, MISSOURI

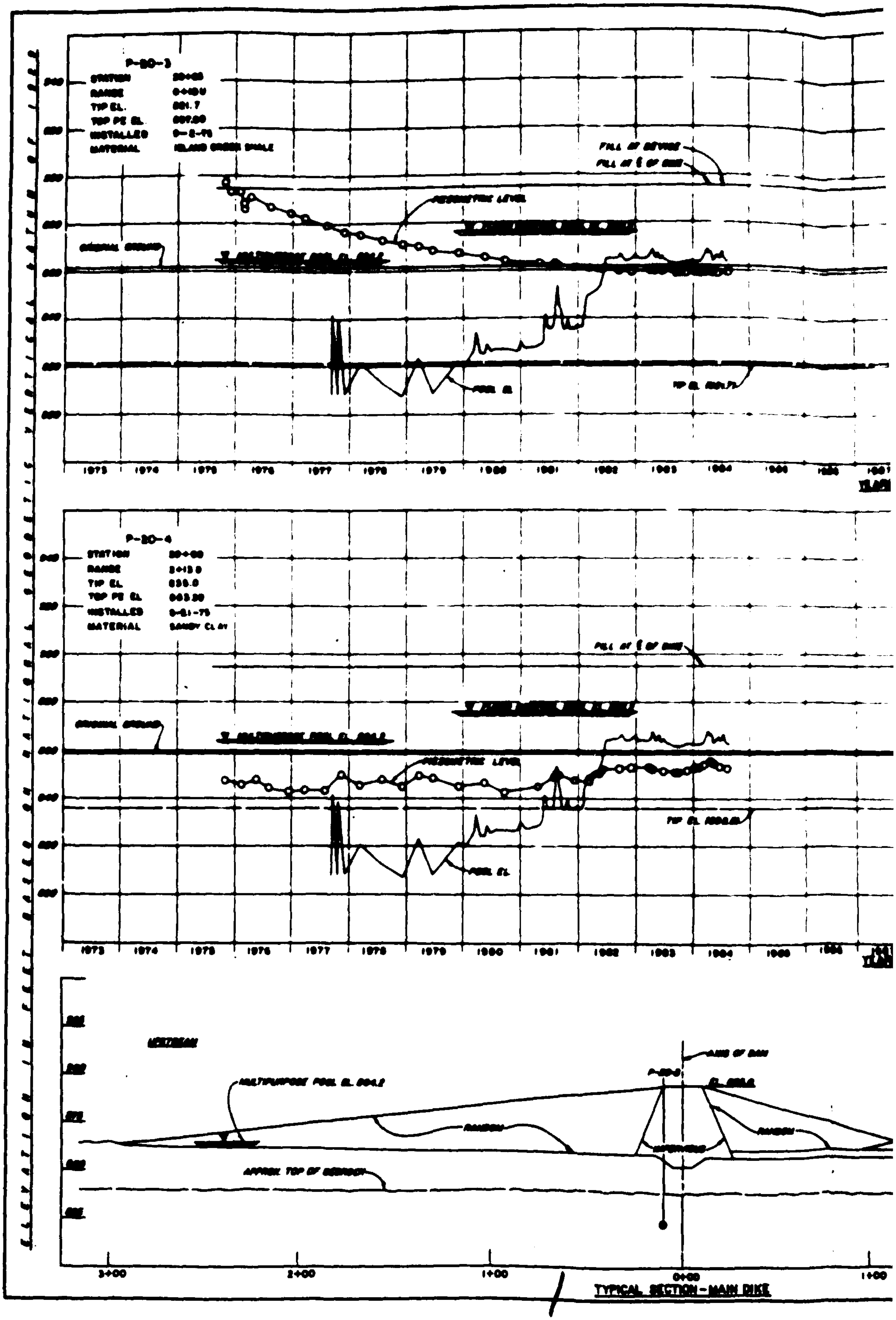
SMITHVILLE LAKE

PIEZOMETRIC LEVELS
P-20-1 (OPEN TUBE)
P-20-2 (OPEN TUBE)

In 1 sheet   Sheet No. 1   Scale: as shown

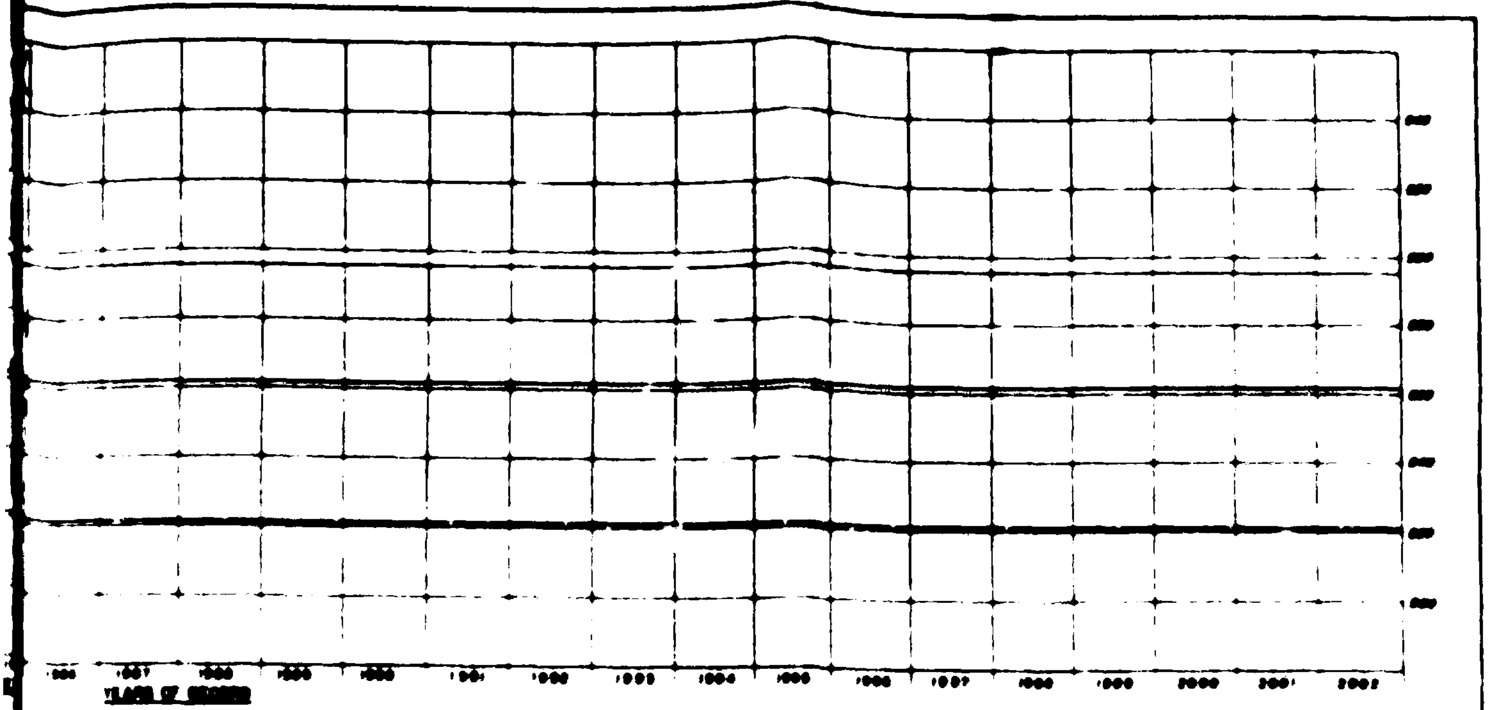
CORPS OF ENGINEERS   U. S. ARMY
KANSAS CITY DISTRICT
FILE NO. [redacted] RP-3-1747
MARCH 1981

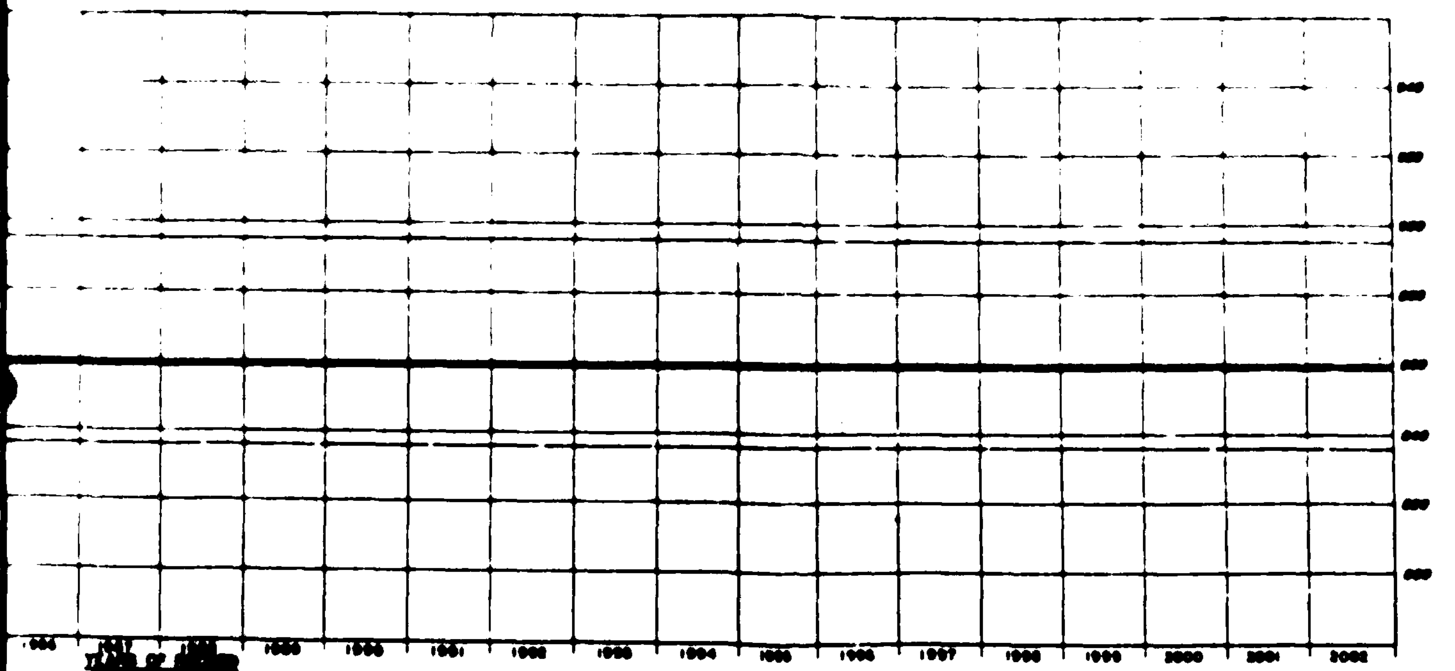
APPENDIX B   PLATE NO. 27

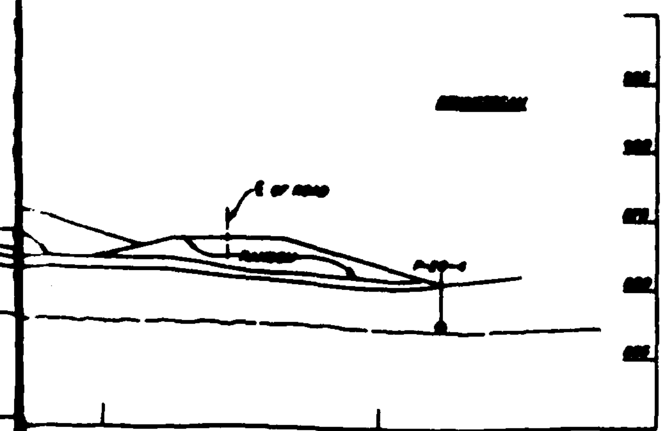


P-20-3
STATION
RANGE
TIP EL.
TOP PE EL.
INSTALLED
MATERIAL
ORIGINAL GROUND
PIEZOMETRIC LEVEL
POOL EL
1973
1974
1975
1976
1977
1978
1979
1980
1981
1982
1983
1984
P-20-4
STATION
RANGE
TIP EL.
TOP PE EL.
INSTALLED
MATERIAL
SANDY CLAY
ORIGINAL GROUND
PIEZOMETRIC LEVEL
POOL EL
UPSTREAM
AXIS OF DAM
MULTIPURPOSE POOL EL.
RANDOM
APPROX. TOP OF BEDROCK
3+00
2+00
1+00
0+00
1+00
TYPICAL SECTION - MAIN DIKE

15-1-7







REVISED SEPTEMBER 1984
LITTLE PLATTE RIVER, MISSOURI

SMITHVILLE LAKE

PIEZOMETRIC LEVELS
P-20-3 (OPEN TUBE)
P-20-4 (OPEN TUBE)

In 1 sheet | Sheet No. 1 | Scale: as shown

CORPS OF ENGINEERS U.S. ARMY
KANSAS CITY DISTRICT
FILE NO RP-3-[redacted] 1748
MARCH 1981

APPENDIX B PLATE NO. 28